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Revolving Sprinkler and Dosing Chamber, Sewage Treatment Plant, Pontiac, Mich.

How to Keep Sewage Filters in Operating Order

Successful Operation of Filters Can Be Secured Through a Complete Understanding of the Functions of the Filter and a Clear Knowledge of the Filter and Its Component Parts

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THE American Society of Civil Engineers and the American Public Health Association a few years ago adopted definitions of terms used in sewage treatment and define the two types of sewage filters as follows:

Intermittent Filter: A natural or artificial bed of sand or other fine grained material to which sewage is intermittently applied in doses and through which it flows, opportunity being given for filtration and also oxidation of the organic matter by bio-chemical agencies.

Trickling Filter: An artificial bed of coarse material, such as broken stone, clinkers, slate, slats, or brush, over which sewage is distributed and applied in drops, films, or spray, from troughs or drippers, moving distributors, or fixed nozzles, and through which it trickles to the underdrains, giving opportunity for organic matter to be oxidized by bio-chemical agencies.

The main purpose of each filter is the oxidization of the organic matter in the sewage, and the filters should be so constructed as to furinsh the best conditions obtainable for the bio-chemical agencies to perform their functions and to thrive.

Aerobic bacteria, upon which this oxidation depends and which is supplied by nature, require a home, which, in this case, is the surface of the sand or stone particles, and food, which is the organic matter in the sewage.

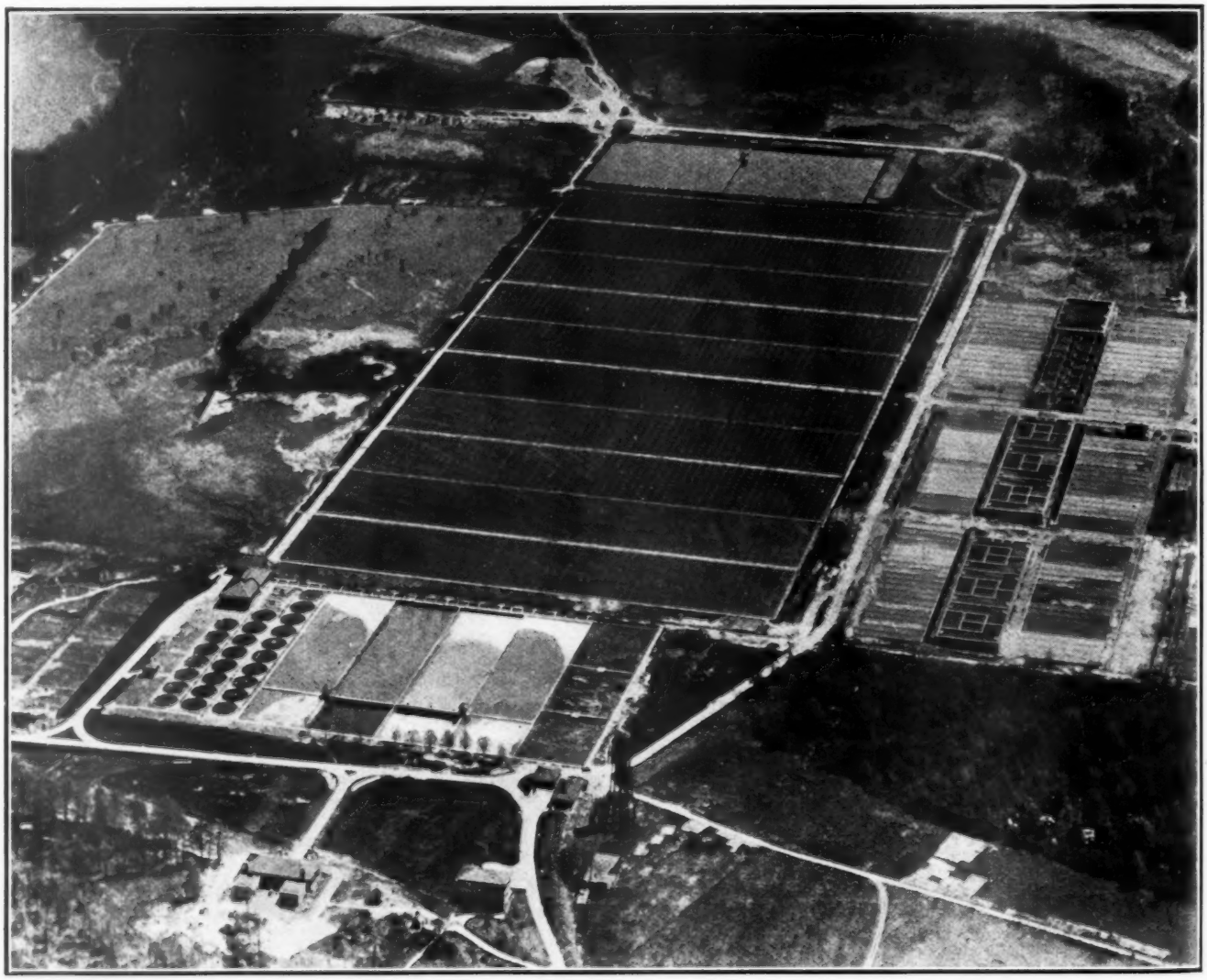
Many years of experience throughout the world have practically determined the various dimensions and capacities of both types of filter beds for various conditions, together with the most effective sizes of materials to be used in them. Granting that either type of filter bed is properly designed for the purpose intended, an equally important factor in results is the proper care in operation of these beds. No filter bed will function properly and efficiently unless it is properly cared for at all times, and the best of filters will soon be failures if neglected.

Intermittent Filter.—The artificial intermittent filter is constructed with underdrains rather close together, with a dirt ridge between each row of underdrains. The underdrain tile are covered with small gravel, the main purpose of which is to prevent the sand from washing into the underdrains. Over this gravel is spread from 18 to 24 in. of clean graded sand. The sizes of these sand grains and the cleanness of the sand are vital factors and most state boards of health specify cleanness, effective size, and uniformity coefficient permissible in this sand filter. A generally recognized dose of sewage, amounting to a 3-in. depth of liquid over the surface of the sand, is usually

designed for. This dose is conveyed onto the intermittent filters through surface pipes or troughs, carrying the liquid to all parts of the sand surface. It is important that the surface of the sand beds should be level and that the places where the liquid first strikes the sand surface should be protected from erosion by gravel or other material. After this dose occurs on any bed, it is advisable to give this bed a rest period of several hours to permit the sewage to seep on through the sand bed and provide a sufficient interval of time for the proper increase of the bacteria, which have been greatly depleted in their work of stablizing the sewage which has passed through it. It is obvious that the clogging of the surface of the sand will prevent this ideal condition; hence, the most vital operating need of the intermittent sand bed is a clean surface. No vegetation should be permitted to grow upon this sand surface and as the strainings of the solids carried over from dosage tanks are trapped in the upper thin film of sand, it is also necessary to keep this film well removed.

A small amount of work spent frequently on a sand bed is much more important than a great deal of work at infrequent intervals. In winter, it is necessary to provide ridges or piles of material at short intervals upon the sand surface, which will support a thin ice covering and permit the liquid to flow underneath this ice film during extremely cold weather. Some plant operators still build these ridges by furrowing the surface of the beds; others start in the early fall and rake up a thin amount of the upper surface with the solids attached in cone-shaped heaps, about 6 ft. apart. This latter practice would seem to be better, inasmuch as the level surface is not disturbed and when the winter is over, these heaps are easily removed without mixing any of the dirt deeper into the sand. With the furrowing system, it is necessary and important to carefully scrape all the upper dirt from the furrow before the sand is re-leveled, and even then, there is a grave danger of mixing the dirt into the upper few inches of the sand bed, which eventually will cause clogging and will necessitate the removal of this depth of dirty sand. If the operator has clearly in mind the need of maintaining a clean bed of sand at all times and never disturbs the surface below a very shallow depth, he should experience no serious difficulty in securing excellent operation. In the course of a year, some amount of the sand bed is necessarily removed from the surface and when this amounts to as much as from 4 to 6 in., the upper surface should be scraped clean and additional proper sand added.

Properly operated, the sand filter bed is probably the most efficient means of sewage purification today, although the operating cost may be somewhat higher than that of the stone beds. Neglect or other factors may cause a sand bed to become so clogged throughout its depth that renewal of the sand becomes necessary. If such a condition exists, a study should be made as to the relative economy between removing and washing the sand as against the removal of the sand and replacing the same with new sand. The tile underdrains may fail and the operator should dig down and investigate the underdrains, should they appear to be



Back River Sewage Treatment Works, Baltimore, Md., Showing Trickling Filters in Center

sluggish, and replace defective tile where needed.

The practice of some operators in by-passing their sand filters during cold weather is an admission of lack of intelligent operating ability on their part. Freezing of underdrain tile may cause many of the tile to disintegrate.

While sand filters may not perform as efficiently in cold weather as in warm, ice-covered outlet streams demand the best purification possible, if fish life is to survive.

All openings in surface distributors should be so adjusted that a uniform amount of sewage flows from each one.

Trickling Filter.—Inasmuch as the total surface area required for trickling filters is not over one-tenth that of the intermittent filter, the floor or bottom of the stone bed is generally constructed of concrete, as well as the side walls, and the expense of a superstructure over the entire bed is often justified. The underdrain tile, quite closely spaced, should drain into a common outlet and have a gallery at each end for inspection and cleanout purposes. Over these underdrains are usually placed 6 ft. or more of hard stone, generally varying from 1 to 3 in. in size, although some filters have smaller stone and others somewhat larger. The sewage is distributed over this stone bed in such a manner as to secure an even amount of liquid over every square foot of surface, if possible. The general custom in America is to accomplish this by means of spray nozzles from dosing or equalizing tanks on the lines of distribution to insure a varying head which tends to secure an even distribution over the surface. Because of the coarser material used and because of the rapidity with which the bed can empty itself of a dose, the rest period as needed in the intermittent filter, is not required with the trickling filter, and many authorities now consider that a very short cycle between doses, even as low as five minutes, may be highly desirable. The important thing in the operation of a properly designed and constructed filter is: First, keep the nozzles clean so the sewage may be distributed evenly over the entire surface. Matches, cloth, and other material may get through the clarifiers and catch in the nozzles or distributors. In keeping these clean, the operator should be careful in handling his particular distributor so as not to damage the mechanism of distribution. Screw and "key" nozzles are easily bent in removing and may seriously interfere with the distribution intended. Occasional removal of the nozzles at the ends of the distribution lines for a short flushing period may be advisable in some instances. Second, watch for ponding on the surface of the bed and stop this ponding as soon as it appears. Third, see that underdrains are clean.

There are various ways in which ponding may be relieved, some of which should be successful with any individual plant. Some operators find the use of a hose in washing down the surface stone breaks up the ponding. Others find that stirring of the upper surface of the stone stops this. Still others find that resting of the bed from dosage for a one-day period often relieves this situation; still others find that the use of chlorine in the dosing chamber for a short period of time will clear up the surface of the beds from ponding. Each operator can determine for himself the needs of his particular stone bed upon trial, and should, of course, use the easiest and most inexpensive method first until he has determined the one best suited to his particular bed.

The underdrains should be accessible for observation and cleaning, and if any of them show evidence of clogging, they should be kept clean at all times. The uniformity of underdrain flow in the underdrains may indicate the uniformity of distribution over the surface of the bed.

The writer's observation over a period of many years convinces him that those sewage plants operated or cared for by men who have a mental picture of the construction of their plant, together with an understanding of the functions of the various units of his plant, are generally the plants which produce satisfactory results, and the poorly operated plants (which are many) are generally operated by men who do not thoroughly understand what they are to do or why they are to do it.

It is to be hoped that the day is not far distant when the vital importance of intelligent sewage plant operation will be recognized by the people and laws passed to secure this much needed condition, or existing laws properly enforced.

Acknowledgment.—The above is a paper presented Oct. 23 at the 11th Iowa Sewage Treatment Conference at Iowa State College, Ames, Ia.

Advantages to Householder of Water Softening

An interesting outline of the advantages of softened water to the average householder was given in Vol. 93, 1929 Transactions of the American Society of Civil Engineers, by C. W. Foulk, Professor of Analytical Chemistry, Ohio State University, Columbus, O. He stated that the Columbus plant removes from 180 to 190 parts per million of hardness from the Scioto River water. This, if left in the water, would destroy about $6 worth of soap per year in the average household. (The household price of soap is taken at 10 ct. per pound.) On the basis of 50,000 households in Columbus, $300,000 worth of soap would be wasted each year. This waste is eliminated by the softened water.

Considering the positive gains arising from the use of soft water, attention is called to the following items in the construction and operation of a house, which are needed in a hard-water city, but not in one with a soft-water supply:

100 bbl. cistern	$110
Electric water pump	120
Installing pump	20
Extra plumbing	50
Total	$300

This means that it costs $300 less to build a house in Columbus than to build the same house under the same conditions in a hard-water city. If interest and depreciation are calculated, it is seen that there is an annual saving of from $25 to $30, because of the soft-water supply.

Long Sewer Outfalls in California

In the earlier outfalls in Southern California, the length from shore was set arbitrarily, even in the case of large outfall sewers, such as that for Los Angeles. The lengths of the more recent outfall sewers have been gauged from the behavior of B. Coli fields of pollution determined from pre-existing outfalls. The outlets are generally made to discharge from upturned ells lying on the ocean floor, in order to avoid their becoming clogged with shifting sand, and to prevent the transmission of water hammer. In a discussion in the September, 1929, Proceedings of the American Society of Civil Engineers, R. F. Goudey, Resident Engineer at Los Angeles of the California State Department of Public Health, gives the following examples of long outfalls in California that have a vertical discharge:

Long Outfalls with Vertical Discharge

City	Population, present	Treatment	Diameter, in inches	Outfall Length, in feet	Depth, in feet
Los Angeles	1,250,000	Screening	84	5,400	63
Orange County	60,000	Screening	42	2,700	40
Santa Monica*	50,000	Screens	24	2,000	25
Santa Barbara	40,000	Screening	42	3,694	40
Ventura	20,000	Screening	17	2,000	30
Venice	20,000	Screening	14	1,360	25
Santa Cruz	16,000	Screening	2-15	2,000	37
Watsonville	10,000	Screening	16	1,500	20
Oceanside	5,000	Septic tank	11	2,000	33
Mission Beach	5,000	Septic tank	14	1,200	22
Pismo	3,000	Septic tank	11	1,700	17
Pacific Beach	3,000	Septic tank	14	1,500	15
Laguna Beach	2,000	Septic tank	10	600	15

*Downward discharge.

The Consumption of Water and the Effects of Metering

Maximum and Minimum Per Capita Consumption for Various Purposes—Comparative Growth of Metering—Effect of Meters on Water Consumption—Reduction in Consumption Due to Metering

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THE path followed in the study of water supply systems extends far back to antiquity, and it is indeed a fascinating story to follow through the ages. Ancient wells are still furnishing water in Egypt and India; some of the water tanks at Aden in Arabia, built probably more than 2,000 years ago are still used in supplying the modern city with water. Athens, Carthage and Loadicea and other ancient cities had their collecting and distributing systems, but undoubtedly the most famous system was that of Rome which is said to have supplied according to various estimates, from 50 to 400 gal. per capita daily around 100 A. D.

From those figures to that of Paris in 1550—1 quart per capita per day—is a far cry, and the difference probably explains why the ravages of pestilence were so widespread in mediaeval days. Even in 1700 Paris was supplying only 2½ quarts of water per capita per day. Since then, with the development of modern collecting and distributing systems the increase in water consumption has helped immensely in raising the standard of living and in making possible the growth of our large cities. At the present time in the United States there are more thas 9,000 communities with water supplies and the domestic consumption ranges from 15 to 75 gal. per capita daily.

The amount of water consumed involves a statistical study of great importance to the water works man in planning future changes and enlargements, and it is the purpose of this article to give briefly an account of the influences that have bearing on the quantity of water that must be supplied to satisfy the demand.

The actual consumption of water is determined in a number of different ways. One common method is to compute it from the number of strokes of the pumps, knowing the piston displacement and applying a correction factor for pump slip. Weirs are sometimes used, especially in the case of gravity systems; but probably the best results are obtained by using some sort of a meter. Irrespective of the manner of obtaining the figure, however, this total consumption includes all the water sent into the mains, including waste and leakage as well as water actually used.

Table II—Maximum and Minimum Per Capita Consumption of Water

Use	Gallons Per Capita Daily Minimum	Maximum	Average
Domestic	20	50	35
Commercial and industrial	10	50	30
Public	5	15	10
Loss	15	40	25
Total	50	155	100

The study of water consumption naturally begins with the division of the supply according to the major purposes for which it is used. Such a classification is of course, useful in laying out different parts of a distribution system, as for instance when an extension is to be made in a manufacturing or in a residential district. For this analysis it is customary to divide the total supplies into five classes, as shown in Table I (Jour. Am. W. W. Assoc. [1915] 181) where the consumption is divided according to the actual figures of a number of cities of different sizes.

In the table are listed only cities in which the consumption of water is wholly metered or very nearly so, as obviously any other method of obtaining consumptions would result in nothing more than estimates. As may be seen there are wide divergences between the various cities in the amounts of water used by the various cities and a discussion of the various factors responsible for these differences is in order.

COMPARATIVE GROWTH OF METERING IN SEVENTEEN CITIES

City	Year	Number Meters	Year	Number Meters	Year	Number Meters
Fall River	1900	6,544	1905	7,523	1928	13,000
Woonsocket	1905	2,366			1927	5,350
Lawrence	1904	5,744			1928	10,500
Hartford	1904	9,860	1919	15,451	1929	19,000
Kansas City	1905	11,344			1929	71,000
Yonkers	1905	6,207			1929	14,600
Lowell	1905	7,889			1928	14,500
Brocton	1905	5,851			1929	10,800
Chicago	1890	3,924	1919	26,000	1929	65,000
Cleveland	1904	30,226			1928	65,000
Syracuse	1904	11,059	1919	28,133	1929	36,000
Covington	1904	5,441			1928	12,600
Springfield	1904	3,987	1919	14,600	1929	21,000
Fetchburg	1905	5,068			1927	56,000
Madison	1903	3,336			1928	9,400
Taunton	1904	2,199			1927	6,000
Rochester	1905	14,065			1929	58,500

Domestic Consumption. — The domestic consumption of course, depends largely on the personal habits of the people of the community, and to a considerable extent the water used is an indication of the standard of living. The following figures, averaged from the statistics of 14 cities present the domestic use in different types of dwellings (Jour. N. E. W. W. Assoc. 27, [1913] 55):

	Gal. per capita per day
Apartment houses	62
First class dwellings	54
Middle class dwellings	34
Lowest class dwellings	15

It is worth noting in this connection that the amount of water used is increasing, even in metered communities.

A decidedly variable item in the domestic consumption figure is the water used for sprinkling lawns and gardens; in dry localities, of course, more water is used in this way than in places where

Table I—Consumption Per Capita for Various Purposes

City	Population	Per Cent Taps Metered	Consumption in Gallons Per Capita Per Day: Domestic	Industrial		Commercial	Public	Loss	Total
Milwaukee, Wis.	430,000	99.4	33.0*	430		.....	5.8	29.2	111.4
New Orleans, La.	360,000	99.7	16.8		12.8		3.2	24.4	57.2
Rochester, N. Y.	250,000	99.5	31.3	18.3		12.2	4.8	28.4	95.0
Fall River, Mass.	115,400	97.8	18.0*	6.8		.....	14.2	7.2	46.2
San Diego, Cal.	85,000	100.0	36.0	7.2		12.0	10.0	15.4	80.6
Wilkinsburg, Pa.	80,000	99.0	25.4		59.6		2.4	12.6	107.0
Lexington, Ky.	40,000	100.0	16.9	17.9		16.0	8.4	2.9	62.1
Oak Park, Ill.	26,000	100.0	52.5	8.5		1.6	2.5	3.9	69.0
Elyria, Ohio	16,000	100.0	38.8	47.0		7.2	5.6	22.5	121.1

*Includes commercial use.

Table III—Effect of Meters on Water Consumption

Per Cent of Services Metered	CITIES 5,000 TO 25,000 POPULATION		CITIES 25,000 TO 100,000 POPULATION		CITIES 100,000 POPULATION AND OVER	
	Number of Cities	Gallons per Capita per Day	Number of Cities	Gallons per Capita per Day	Number of Cities	Gallons per Capita per Day
0–10	60	161	16	198	3	218
10–20	20	199	3	144	3	149
20–30	17	128	2	208	3	154
30–40	15	108	4	116	3	170
40–50	8	88	5	105	3	160
50–60	18	141	5	165	0	0
60–70	18	120	4	91	0	0
70–80	16	140	10	84	4	97
80–90	31	79	9	107	10	127
90–100	145	91	53	94	22	97
Total	348	.....	111	.....	51	

Table IV—Effect of Meters on Water Consumption

Group	Number of Cities	Population Per Group	DAILY CONSUMPTION			Number of Services	Number of Meters	Per Cent Metered
			Total Millions	Gallons per Capita				
				Range	Average			
1	4	379,198	18.60	45–55	49	43,502	41,109	94
2	8	1,307,341	85.88	56–70	66	193,057	173,410	89
3	8	926,252	71.23	71–80	77	149,781	122,165	81
4	13	1,096,729	97.03	81–95	88	170,882	128,571	75
5	7	1,083,068	112.33	96–110	104	180,039	125,271	69
6	7	1,408,465	159.74	111–120	113	238,291	144,497	60
7	8	1,588,716	203.72	121–130	128	271,327	64,266	24
8	9	1,491,518	206.49	131–150	138	291,517	55,450	19
9	4	1,993,951	351.66	151–200	176	432,583	76,100	17
Total	68	11,275,238	1,306.68	45–200	110	1,970,979	930,839	47

the rainfall is heavier. For sprinkling lawns 7 to 8 gal. of water are required per 100 sq. ft. of surface and inasmuch as lawn sprinklers and garden hose nozzles will pass from 225 to 660 gal of water per home (Jour. N. E. W. W. Assoc. 28 [1914] 377), the amount of water used in this manner will be considerable; in Denver, 10 per cent of the total supply is said to be used in this way.

High distribution pressures have a very direct bearing on domestic consumption. Hartford, Conn., for instance, saved 300,000 gal. daily or approximately 4 per cent of its total demand by throttling its valves. (Eng. Rec. Feb. 5, 1916, p. 184.) The introduction of sewers will increase the use of water in some instances as much as 100 per cent; incidentally it has recently been suggested that the cost of extensions to sewer systems be apportioned among the properties using them according to their respective water consumptions as determined by meter readings.

Industrial Consumption. — Industrial requirements for water vary with the nature of the industries served; tanneries, laundries, chemical works, and paper mills are large consumers, while water for steam generation is a considerable item in most industrial demands. In some localities industries draw their own water from rivers or from their own wells and there is practically no industrial demand, while in some cities the industrial load is more than half the total water consumed. Commercial users of water embrace not only stores and hotels but also rooming and apartment houses.

Water for public uses is generally furnished free and unmetered though often the amount of water so used is considerable—13.6 per cent of the total consumption in San Diego, Cal., and in Lexington, Ky., in 1914 (Proc. Am. W. W. Assoc. [1915] 191-195). It includes water for schools, public buildings, parks, street sprinkling, road construction and fire fighting.

The last item, "loss," is sometimes called "unaccounted for" or perhaps more aptly "waste." It covers leakage from mains and fixtures, under-registration of meters and careless or wasteful use for purposes legitimate in themselves. Metering will minimize the last named waste and any persisting waste will be metered and paid for. In a meter system, the total loss should not be greater than 20 per cent of the whole consumption; the loss then would be largely due to leakage from mains and careful inspection will keep this at a low figure.

As averages of the above figures, Turneaure and Russell (Public Water Supplies p. 22) give the figures shown in Table II, applying to supplies having good metering systems.

Minimum or maximum sizes for all purposes are not likely to occur at the same time, so that the totals of minima or maxima are extreme figures. Probably a normal minimum could be 60 gal. per capita daily while a normal maximum would be close to 125 gal.

In unmetered system the normal total consumption would not be far from 150 gal. per capita daily as against an average 100 gal. per capita in metered systems. Under especially favorable conditions, the consumption in the unmetered system might use as high as 300 gal. per capita daily; for example, Chicago in 1921 was using 276 gal. per capita daily while Coeur d'Alene, Idaho, used 316. (Flinn, Weston and Bogert, Waterworks, Handbook, p. 578.)

Variations in Consumption.—The figures given above show the average daily demands over a long period of time, but there are variations, sometimes of considerable magnitude, existing over short periods. Of course, all parts of the supply and distribution systems must be made large enough to supply the greatest demand that will be imposed on them and it is consequently necessary to study these variations.

The usually accepted figures are that the maximum monthly rate will be 125 per cent of the average, while the maximum daily rate will be 150 per cent of the average; for smaller cities, however, the maximum daily rate is likely to be over 200 per cent of the average rate. Further, the maximum hour will draw water at 150 per cent of the rate of the maximum day. Consequently the system should be capable of delivering 150 per cent of 150 per cent, or 225 per cent, of the average consumption. If the system is also to be used for supplying water for fire fighting, it should be capable of handling in addition 150 to 200 per cent of the average. In large cities, incidentally, the fire consumption should be about 50 per cent of the average, while in very small towns it may be as high as ten times the average rate.

REDUCTION IN CONSUMPTION DUE TO METERING IN CITIES THAT HAVE ADOPTED A UNIVERSAL METERING PROGRAM

City	Universal Metering Adopted Date	Per Cent of Services Metered				Per Capita Consumption				Population Time of Adoption	Estimated Population July 1, 1925	Per Cent Increase	Maximum Rate Per 1,000 Gallons Cents
		5 Years Previous	Time of Adoption	5 Years After	1925	5 Years Previous	Time of Adoption	5 Years After	1925				
Detroit	1913	10.0	23.6	97.4	99.07	162.0	163.0	144.0	130.0	624,139	1,257,634	101.0	8.7
Cleveland	1901	4.9	68.0	100.0	100.0	168.9	130.7	96.1	127.4	399,658	912,502	128.0	8.0
Los Angeles	1905	0.0	19.1	55.4	99.0	300.0	175.0	141.0	119.0	210,838	705,411	234.0	9.7
Milwaukee	1899	61.0	83.0	95.0	99.0	101.0	83.0	86.0	128.0	277,231	492,087	77.0	9.3
Newark	1915	49.0	60.6	92.2	95.5	107.0	110.0	108.2	100.2	380,936	445,606	14.0	19.2
Minneapolis	1905	25.1	45.7	79.6	100.0	92.7	66.5	73.8	107.8	252,063	415,280	65.0	8.0
Cincinnati	1915	32.8	64.9	98.2	98.6	127.8	125.9	123.4	116.1	382,419	407,835	6.5	16.0
Kansas C'y, Mo.	1905	29.0	38.0	44.4	82.5	60.7	93.8	98.0	125.7	206,066	359,650	74.0	22.7
Rochester	1905	28.6	41.0	97.0	99.0	76.2	78.9	80.8	93.9	190,378	325,211	71.0	16.0
Columbus	1905	32.0	76.2	91.31	99.1	150.0	59.6	75.6	92.8	153,530	266,709	74.0	16.0
Oakland	1910	43.6	57.2	100.0	100.0	90.9	85.6	54.3	64.1	150,174	246,893	64.0	27.9
St. Paul	1912	51.1	84.8	97.3	92.5	60.7	60.9	81.2	71.5	218,734	243,946	12.0	12.0
Akron	1912	25.0	45.0	100.0	100.0	106.3	117.2	93.2	87.0	96,941	278,119	187.0	21.4
Av.of 13 Cities since	1901	30.1	54.39	87.5	97.8	123.4	103.8	96.5	104.8	3,543,167	6,358,883	79.0	16.0

METERS MANUFACTURED 1910-25
(five manufacturers)

Year	No. Meters	Year	No. Meters
1910	295,000	1918	340,000
1911	310,000	1919	330,000
1912	340,000	1920	410,000
1913	325,000	1921	390,000
1914	365,000	1922	490,000
1915	375,000	1923	560,000
1916	410,000	1924	575,000
1917	385,000	1925	640,000

Finally, in making provision for future demands on the water supply system, the future increase in population must be estimated; this is done by interpolating past rates of growth of the city and by comparing with the growth of other cities of similar size

making allowances, of course, for the different characteristics of the two cities. Cognizance should be taken as well of the fact already noted, that the average rate of legitimate water consumption is increasing. As systems become more nearly completely metered, however, the rate of decrease of illegitimate consumption will more than compensate for the rate of legitimate increase until the rate of meter installation drops down.

Despite the prejudice against water meters in certain sections and notwithstanding the objections to their installation raised as political expedients it is worth noting that of the municipalities that have adopted them not one has ever gone back to the old system. (Eng. News Record, Feb. 7, 1918, p. 269.)

Reduced Waste with Meters.—Countless numbers of cases where water meters have reduced the waste of water may be cited, and in every case the reduction is very marked. Table III from the report of the Engineering Board of Review to the Board of Water Commissioners of Denver shows clearly that water consumption decreases as the percentage of services metered increases. Another tabulation by Hill (Eng. News-Rec. 79, 1917, 312) groups 68 cities with a total population of more than 11,000,000 according to their per capita consumption, as shown in Table IV; as may be seen the percentage of meters to total services falls rapidly as the consumption rises. As a specific instance the case of Boston shows a rapid decrease in the rate of consumption as the number of meters increases.

With such practical unanimity of opinion as exists on the value of metering and with a realization of the axiomatic equity of the system of paying for water actually received, it is difficult to class those approving the installation of meters as anything except proponents of an antiquated and unfair method of taxation. Just as the gas industry was forced for its own protection to turn from flat rates to meters, so will the water supply industry, both public and privately owned, turn eventually and completely to meters, and that far before the millenium is reached.

Acknowledgment.—The foregoing is reprinted from October Water and Light, the journal of the southeastern section of the American Water Works Association.

Cost of Pumpage at Dubuque, Ia.—The following taken from the 29th annual report of the Water Department of Dubuque, Ia., shows the average cost of pumpage for the year ending March 31, 1929:

	Per 1,000 Gal.
Operating and maintenance only	$0.0662
Operating, maintenance and depreciation	.0914
Operating, maintenance and interest	.0770
Operating, maintenance, depreciation interest	.1062

Some English Notes on Activated Sludge Process

By ARTHUR J. MARTIN

Whenever a new and promising process for purifying sewage makes its appearance people are apt to jump at the conclusion that existing methods are doomed to be discarded. We had an instance of this 30 years ago on the advent of the septic tank, and history is repeating itself in the case of the activated-sludge process. This process has undoubtedly come to stay, but it is far more likely to supplement the older methods than to supersede them.

In the first flush of enthusiasm, too much was expected of the process, and it has been asked to do work for which it was not really suited. A common mistake was to rush the sewage into the aeration tank without adequate preparation. It is now realized that a preliminary settlement is desirable, and our old friend the sedimentation tank is accordingly coming into its own once more.

And it is by no means certain that it is always wise to dispense with filters. An activated sludge tank is very sensitive to change in the composition of the sewage. A sudden rush of liquid trade wastes may kill the organisms in the sludge, in which case it may take months to get it back into condition. The bacterial films in a trickling filter are hardier, and, as Mr. Whitehead, the engineer to the Birmingham, Tame and Rea Drainage Board, points out, the margin of aeration in a filter, and consequently its ability to cope with exceptional states of the sewage, is infinitely greater than given by any activated sludge plant.

Mr. Whitehead's predecessor, Mr. Watson, has drawn attention to another very significant point. The purification effected by the activated-sludge process is at first very rapid, but it soon slows down. Sixty per cent of the impurities of the Birmingham sewage are removed in the first hour; it takes five hours to remove another 30 per cent. A trickling filter would do it in an hour.

Sewage which has been treated for an hour with activated sludge is free from colloidal matter, and is much more readily oxidized than sewage which has merely passed through a sedimentation tank. As a result of this improvement, the quantity which can be purified by each cubic yard of filtering material is more than doubled.

Owing to the rapid growth in the population of the drainage district, the board was faced with the necessity for laying down an additional acre of filters each year to keep pace with the increased volume of the sewage. To obviate this need the board have laid down aeration tanks, or, as Mr. Watson prefers to call them, "flocculation tanks," between the sedimentation tanks and the filters. By so doing they increased the capacity of the filters twice or threefold, and the need for laying down any more beds has been indefinitely postponed. Not only so, but the nuisance which has been experienced from smell and from flies has been absolutely done away with.

Purification Standards.—Before designing works to purify sewage, we must know how far the purification will have to be carried. No question in connection with sewage has given rise to keener controversy than that of the standard of purity which should be aimed at. Opinions differ widely as to the nature of the tests which should be applied to effluents, their severity, and more particularly as to whether the standard should be enforced in all cases.

Various chemical tests have been proposed, framed, we may suspect, with more regard to the convenience of the chemist than to their actual value in practice. The rivers boards which have been set up in certain watersheds have adopted standards of their own, mostly different, which they apply to the effluents discharged into their rivers.

Too often these standards have been regarded as an end in themselves, instead of as a means for safeguarding the purity of the river. Little regard has been paid to the circumstances of individual cases, such as the character of the stream and the relative volumes of the diluting water and the effluents.

Lord Iddesleigh's Commission made an earnest attempt to put an end to this confusion, and recommended a common-sense standard to be applied in ordinary cases. In order to comply with this standard, the effluent must not contain more than three parts per 100,000 of suspended matter, nor absorb in five days more than two parts per 100,000 of dissolved oxygen. This standard, however, was not to be applied indiscriminately, for, as the Commissioners pointed out, the dilution afforded by the stream is the chief factor to be considered. When the volume of the stream, as compared with that of the effluent, is great, the standard should be relaxed, and with the dilution exceeding 50 volumes, all tests might be dispensed with.

All the standards now in force or proposed relate solely to the chemical qualities of the effluent. No one has yet ventured to advocate a biological standard. But it should never be forgotten that a sewage effluent, however highly it may be purified, is not suitable for use as drinking water. For the danger from sewage lies not in its chemical impurities, but in the possible presence of the germs of disease, and these are not destroyed nor removed with certainty by any of the processes in ordinary use. The effluent from a sewage works, however efficient, must therefore be regarded as potentially dangerous.

Acknowledgment.—The foregoing is an abstract of a paper presented Oct. 17 before the Institution of Mechanical Engineers of Great Britain.

Sanitary Aspect of Lake Michigan Diversion Case at Chicago, Illinois

A Non-Controversial Discussion of Chicago's Problem: Historical, Present Sewerage System—Efficiency of Sewage Treatment—The Water Supply—What Filtration Will Accomplish

By L. R. HOWSON

Alvord, Burdick & Howson, Consulting Engineers, Chicago, Ill.

FOR many years, the diversion of water from Lake Michigan by Chicago for sewage disposal purposes has been the subject of bitter controversy, culminating in a suit in the United States Supreme Court started about three years ago. The importance of the issue may be judged by the fact that about two-thirds of the people of the United States are parties in the litigation. The Supreme Court appointed Charles Evans Hughes as special master to take evidence which extended over a two-year period. In January, 1929, the Supreme Court made its initial findings, which required the Chicago Sanitary District to dispose of its sewage by means other than lake water diversion. This decree, when effective, will result in the discharge of part or all of the sewage plant effluents into the lake from which the water supply is taken. The practicability of safeguarding the public health under this program has been the subject of much debate. It is this phase of the case to which this discussion is directed.

A brief historical outline of Chicago's sewage disposal and water supply problems may assist toward an understanding of sanitary problems which the court's finding imposes upon the Chicago district.

Historical.—Chicago and the industrial region adjacent to it occupy the southwestern shore of Lake Michigan. The topography is comparatively level and most of the ground lies at an elevation less than 20 ft. above Lake Michigan. The city started near the mouth of the Chicago River, which was a small sluggish stream with a total drainage area of approximately 307 square miles, being made up of two branches, known as the north and south branches, which unite about one mile west of the mouth of the river.

The first sewers, as well as the first municipal water system, were built in 1856, at which time the city had a population of about 80,000 people. The occupied area was then so located that most of the sewers naturally drained directly to the main branch of the Chicago River and thence discharged directly to the lake. As the growth of the city increased, the expansion took place to the west, north and south, the natural drainage of which was either to one of the branches of the river or directly to the lake. The water supply has been taken from the lake from 1856 to date.

By 1890, the population had increased to 1,209,000 people and at the present time, exceeds 3,500,000 in the Sanitary District.

The discharge of the sewage into the lake within close proximity of the water works intakes resulted in high rates of water-borne disease in the Chicago district, the typhoid death rate reaching a peak figure of 173.8 per 100,000 in 1891.

In 1886, a drainage and water supply commission, known as the Hering, Artingstall and Williams Commission, was appointed to make a complete investigation of the subject of water supply, sewage disposal and drainage for the City of Chicago. This commission advised the construction of the drainage canal with a capacity of 10,000 cu. ft. per second and the construction of intercepting sewers which would divert the entire sewage from the lake and discharge it mixed with water drawn from Lake Michigan into the upper Des Plaines and thence into the Illinois and Mississippi water sheds.

As an outgrowth of this report. the legislature of the state of Illinois in 1889 passed the Sanitary District act and shortly thereafter the people of the district voted affirmatively on the creation of the Chicago Sanitary District.

Plans were prepared for the construction of a channel extending from a connection with the south branch of the Chicago River at Robey St. to a connection with the Des Plaines River near Lockport, a distance of some 28 miles. Construction was undertaken and the canal completed and placed in service in 1901 and since that date, the flow of the Chicago River has been from rather than into Lake Michigan.

The Calumet District to the south and east of Chicago continued to drain into Lake Michigan. In 1911, the construction of a channel known as the Calumet-Sag channel was started to divert the normal flow of the Calumet River from Lake Michigan to a connection with the drainage canal at approximately mid-distance between the Robey St. and Lockport ends of the canal. This channel was designed with a capacity of 2,000 cu. ft. per second. It was completed in 1922 and since that date, the flow of the Grand and Little Calumet Rivers has been away from rather than into Lake Michigan at such times as the stream flow did not exceed the carrying capacity of the canal and its connections.

The construction of the main drainage canal and the Calumet-Sag branch effected a remarkable improvement in the Chicago death rate from water-borne disease and to a large extent removed the sewage nuisance from the streams themselves. This result has, however, been accomplished by transferring the bad conditions in the Chicago and Calumet Rivers to the Des Plaines and Illinois Rivers in which nuisance has been created, all fish life killed and other damage resulted for more than 100 miles downstream. The Des Plaines and Illinois Rivers today are practically an open sewer for 150 miles distance from Chicago.

The important facts with respect to the typhoid death rate in Chicago are as follows:

Average for five years prior to opening of canal reversing flow—34 per 100,000.

Average for five years after opening of canal reversing flow—28.4 per 100,000.

Average for five years after completion of intercepting sewers diverting sewage from the lake—12.1 per 100,000.

Average for five years after the pasteurization of milk and the start of water chlorination—5.9 per 100,000.

Average for five years after complete chlorination of water supply—1.2 per 100,000.

The upper part of the diagram (Fig. 3) shows the typhoid statistics for Chicago for the period from 1890 to date. In this diagram are indicated the dates at which sanitary improvements were placed in service so that their effects may be visibly indicated.

The lower half of the diagram shows for comparison another important typhoid death rate reduction accomplished by water filtration, to which reference will be made later.

On Jan. 14, 1929, the Supreme Court of the United States delivered an opinion with regard to the diversion of water, which denied the right of diverting water at Chicago except that negligible quantity, if any, which might be required for navigation in the Chicago River as a part of the Port of Chicago.

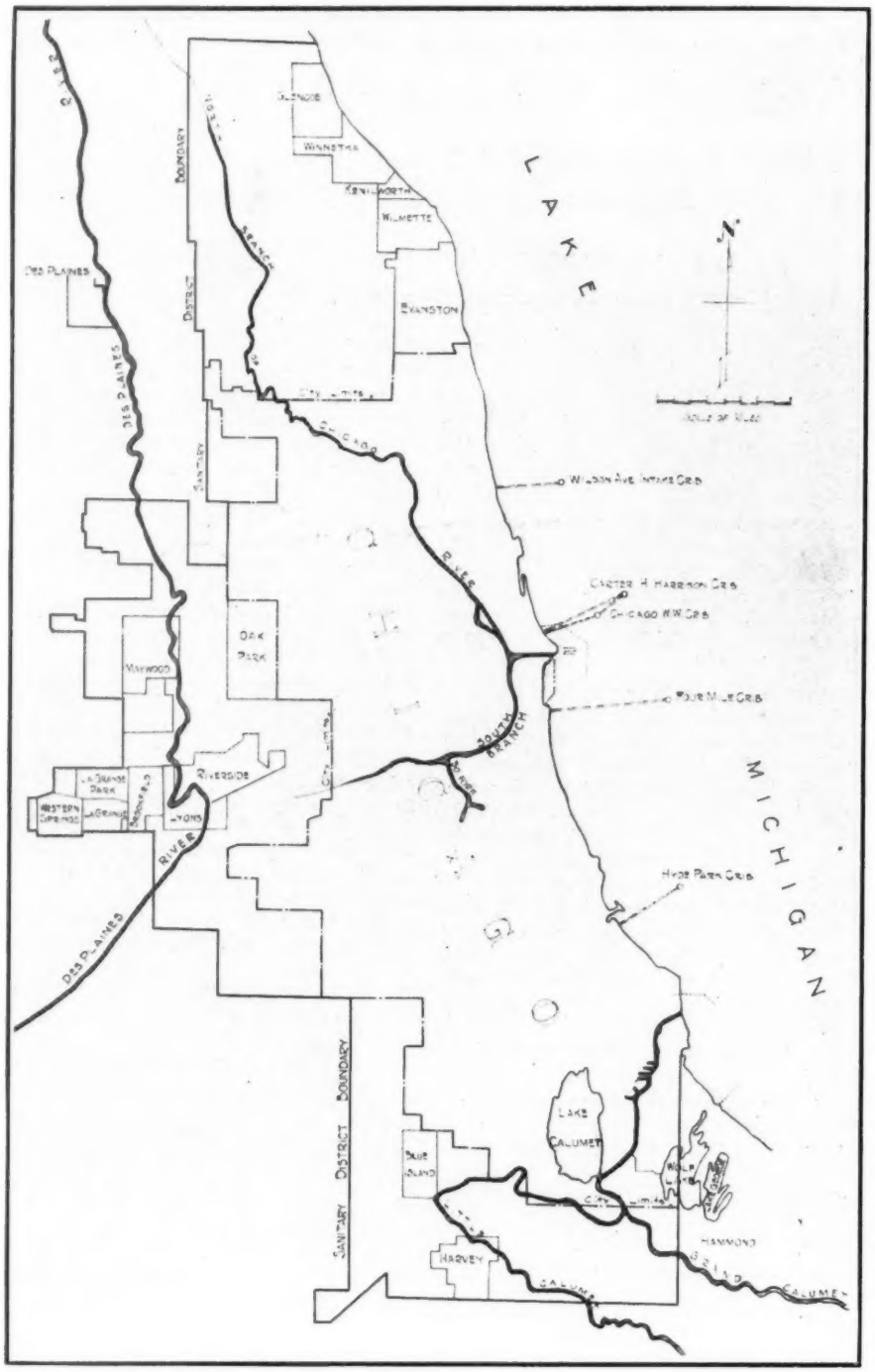


Fig. 1—Drainage Channels in the Chicago Sanitary District Prior to Diversion

In keeping, however, with the principles on which Courts of Equity condition their relief and by way of avoiding any unnecessary hazards to the health of the people of the Chicago section, the Supreme Court decided to so frame its decree as to accord to the Sanitary District a reasonably practicable time in which to provide some other means of disposing of the sewage, at which time there shall be a final permanent operative and effective injunction. The special master, Mr. Charles Evans Hughes, was instructed to take such further evidence as would enable him to advise the court as to:

1st: What negligible amount, if any, of diversion was necessary for navigation in the Chicago River as a part of the Port of Chicago.

2nd: What sewage disposal works would be necessary to care for the sewage of the Chicago district by means other than diversion.

3rd: What period would be required for the construction of such works.

This decree, when effectively carried out, will result in a part or all of the sewage plant effluent and storm water overflow being discharged into Lake Michigan, from which the water supply is taken. The water supply and sewage disposal problems at Chicago are and always have been one problem, though administered by two governmental agencies.

Present Sewerage System.—Sewerage in the Chicago Sanitary District is almost entirely on the combined system, although the Hering, Artingstall and Williams report, made in 1887, recommended separate sewers for the then outlying areas. Had that recommendation been adopted, nearly 80 per cent of the present sewer mileage would have been on the separate system and over 50 per cent of the population would have been tributary thereto. The problem of building intercepting sewers and of caring for storm water overflows would have been much simplified.

The Sanitary District now has under construction a comprehensive sewage disposal program including four major and several minor projects.

The North Side sewage treatment works have been finished. This is an activated sludge plant designed to serve approximately 800,000 people.

The West Side project, to serve 1,915,000 people as of 1945, has been started, originally as Imhoff tank treatment only, later to be followed by sprinkling filters, but since the Supreme Court decision, the Sanitary District has modified its plans for this plant so as to make it an activated sludge plant.

The Southwest side plant, which is to be located directly across the main drainage canal from the West Side plant, has not as yet been started. It is now planned to be an activated sludge plant to serve 1,320,000 people in 1945. The large stockyards organic load will be handled at this plant.

The Calumet project has been designed as an Imhoff tank and trickling filter plant to serve approximately 400,000 people. The tanks have been in operation approximately seven years. The sprinkling filters and secondary sedimentation are now included as a part of the Sanitary District's program to comply with the requirements of the United States Supreme Court.

Efficiency of Sewage Treatment.—While there has been some divergence of opinion, as reflected in the testimony, as to the quality of an activated sludge plant effluent, the operation of the Milwaukee plant, the largest yet in full operation, has demonstrated that the effluent from such an activated sludge plant in the sizes contemplated for Chicago, will be practically clear, odorless, colorless and stable and that the process will have removed at least 95 per cent of the suspended solids, at least 95 per cent of the bacteria, 98 per cent to 99 per cent of the b. coli and 96 per cent of the biochemical oxygen demand. It will contain 5 to 7 p.p.m. of dissolved oxygen and oxygen in the form of nitrates and nitrites up to 15 p.p.m. Based upon the Milwaukee operating experience, the remaining B. O. D. will be about 10 p.p.m. or less. It will be

permanently stable, that is, it will not again putrefy if stored in an open vessel or discharged into a water course in which it constitutes the total flow.

With a combined system of sewers, all of the sewage flow cannot be passed through the disposal plant on account of the relatively high ratio of storm water to sanitary flow. This necessarily results in the by-passing of some sewage during storms. It has been estimated that approximately 1½ per cent to 2 per cent of the sewage flow may reach the river channels untreated during the year, the exact percentage depending quite largely upon the capacity of the interceptors and the proportion of first storm flows which are carried through the plant.

The amount of storm water which could be discharged into the various water courses if all of the sewers were running full is approximately 12,000 c.fs. The interceptors are capable of taking about 4,000 c.f.s. to the disposal plants so that in extreme conditions 8,000 c.f.s. of mixed storm and sanitary flow would reach the streams.

The effluents from the various plants could be chlorinated so as to effect practically 100 per cent of b. coli removal if desired. It is somewhat more difficult to chlorinate the storm water overflow from over 300 separate outlets. All of this flow which may reach the lake, however, has to pass out through either the main Chicago River or the Calumet River and while the applying of chlorine to such large volumes has not been attempted to date, the results obtained in chlorinating raw sewage in large volumes would indicate its practicability. At the Cleveland Easterly plant, where the raw sewage from over 500,000 people is chlorinated, b. coli reductions have averaged over 96 per cent for the past three years and have exceeded 99 per cent when a chlorine residual of about .3 p.p.m. was carried.

So far as the sanitary aspect of the water diversion case is concerned, the question becomes one of the effect of activated sludge plant effluent and storm water overflows primarily upon the water supply and to a lesser extent upon the bathing beaches.

The Chicago Water Supply.—The Chicago water supply is drawn through four intake cribs located from three to four miles off shore and connected by tunnels to the 12 major pumping stations. The water receives no treatment other than chlorination.

About two-thirds of the time, the water drawn from the Chicago cribs is clear. It is often quite pure for short periods. However, its sanitary quality cannot be depended upon and for a considerable portion of the time at all of the intake cribs, it is distinctly bad. The water as delivered to consumers is often decidedly turbid for weeks at a time.

During 1927, the turbidities varied from 1 to 115 p.p.m. with a yearly average of 11.5 p.p.m. for all stations and 17.8 p.p.m. at the 68th St. crib. The water from the 68th St. crib had an average turbidity for a month of 50.4 p.p.m.

The net result from a sanitary standpoint is that the water as chlorinated has been good. The taste is frequently objectionable and the turbidity exceeds 10 parts per million on the average approximately 30 per cent of the time.

Filtration of the present water supply has been recommended by all conversant with the problem for a number of years. Filtration, however, carries with it the prerequisite of metering, for at the present time Chicago is pumping approximately 300 gal. per capita per day and the cost of filtration plants, sufficient to care only for the water wasted, would approximate $30,000,000. It has been estimated that if Chicago were to install meters on all services, in a 10-year period, there would be saved in the next 25 years $233,000,000 in operation and construction costs. In addition, it has been estimated that approximately $50,000,000 would be saved in the cost of intercepting sewers and sewage disposal plant construction. With complete metering, the quantity of water pumped, which it is assumed

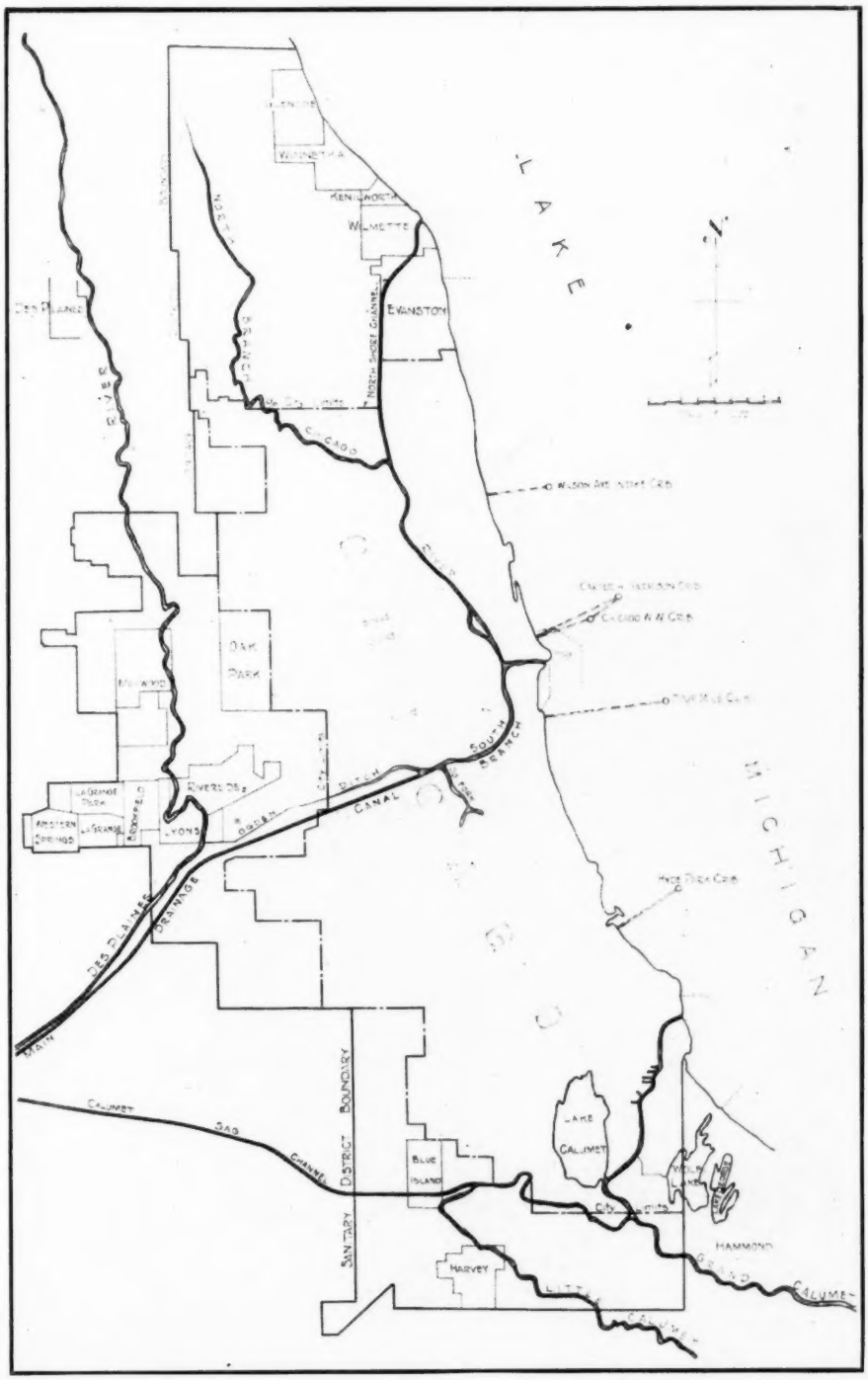


Fig. 2—Drainage Channels in the Chicago Sanitary District After Diversion

is substantially the same as the sewage flow, will be approximately 1,000 cu. ft. per second between 1935 and 1940.

What Filtration Will Accomplish.—Filtration has demonstrated its ability to treat highly polluted waters satisfactorily. Among the notable examples are Niagara Falls and the Ohio River cities (Fig. 3).

Niagara Falls, whose water supply is polluted by the fresh sewage of Buffalo and environs, was one of the most important typhoid centers of the country, prior to filtration in 1913. The typhoid death rates were as high as 185 per 100,000 and averaged 125.8 per 100,000 for the 14 years preceding filtration. Following the start of filtration there were 18 months with no deaths from typhoid and the average typhoid death rate since filtration has shown a reduction of over 95 per cent from that prevailing prior thereto.

On the Ohio River city after city, from Pittsburgh to Cairo, takes its water supply from the river, returning the sewage again to the stream, the water being used again and again by the cities downstream after filtration. The b. coli index of the raw Ohio River water frequently exceeds 100,000. Filtration and chlorination produces from it a safe water supply.

In addition to the bacterial removal filtration will, of course, remove the turbidities now prevailing in the Chicago water supply.

Filtration will insure a clean water for Chicago 365 days in the year. Filtration should be installed at Chicago regardless of whether the diversion from the lake is zero or 10,000 c.f.s.

Chicago's Problem.—The Supreme Court having already decided that all diversion, except such negligible amount, if any, as may be required for navigation in the Chicago River, shall cease, the Chicago district is confronted with the necessity for devising such means of caring for its sewage and of protecting and purifying its water supply as will adequately safeguard the public health.

Among the measures available to accomplish this result, are the following:

1. The treatment of all sewage other than storm water overflow through the most efficient type of treatment plants which experts on both sides have agreed to be the activated sludge process.

2. The chlorination of the sewage disposal plant effluents.

3. The chlorination of all storm water overflows by carrying a residual chlorine content of approximately .3 part per million in the liquid flowing into the lake through the Calumet and Chicago Rivers.

4. The metering of the water supply which will make filtration practicable.

5. The filtration of the public water supply utilizing the present intakes and standard filtration methods.

6. The abandonment of those intakes nearest to the mouth of the rivers and the substitution therefor of a super-intake located approximately 20 miles to the northeast of the mouth of the Chicago River.

7. The modification of the standard filtration processes by double coagulation as is extensively practiced on the highly polluted Ohio River supplies or the adoption of excess lime treatment.

8. Sanitary control and sterilization of the bathing beaches.

9. Should the Supreme Court allow a diversion from the lake for navigation purposes in the Chicago River equivalent to the water supply and the run-off from the Chicago River watershed, it would be practicable through the operation of controlling works located at the river mouth or at the head of the canal to prevent any outflow of either sewage plant effluent or storm overflow to the lake.

Opinion as expressed in the hearings before Mr. Hughes has differed as to the measures or combination of measures which might be necessary to adequately safeguard the public health and the writer does not intend in this paper to discuss these controversial matters. It must, however, be apparent that if sanitary engineering science has not sufficently developed to safeguard the health of the Chicago community through a combination of sewage treatment and water purification works, a large part of the population of this and other countries is destined to live under unsanitary conditions. Chicago, in contrast to Milwaukee, Cleveland, Toledo and other important lake cities, is the only one at which diversion from the lake was a practicable possibility. Can anyone successfully argue that Cleveland or Milwaukee must cease to exist because of the necessity for living under conditions which are now to be imposed upon Chicago? The facts are that Cleveland's typhoid death rate with water filtration but with only 10 per cent to 15 per cent sewage treatment and Milwaukee's with complete sewage treatment, but with chlorination only of its water supply, are substantially the same as Chicago's under the present conditions of diversion from Lake Michigan. Certainly, Chicago with proper treatment of its sewage and with adequate purification of its water supply can continue to function and expand without detriment due to the decision of the Supreme Court.

Acknowledgment.—The foregoing is a paper presented at the recent meeting of the American Society for Municipal Improvements.

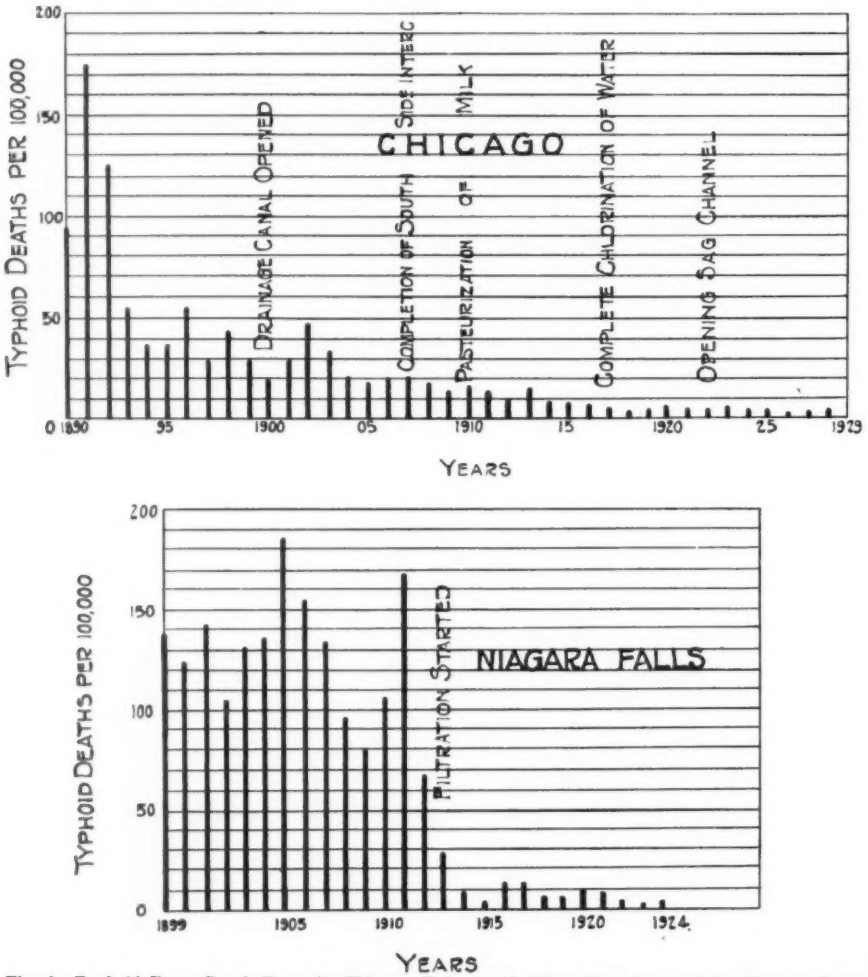


Fig. 3—Typhoid Fever Death Rates in Chicago, Before and After Diversion, and in Niagara Falls Before and After Filtration

Recreational Use of Reservoirs

Practice of the Department of Water and Power of the City of Los Angeles, California

By CARL WILSON

Laboratory Director, Department of Water and Power, City of Los Angeles, Calif.

AMONG the numerous phases of sanitation in connection with water supplies the question of recreational use of reservoirs impounding water intended for domestic consumption is one of perennial recurrence, seemingly incapable of final solution, at least for certain social elements.

From Cloud to Kitchen Tap.—First of all, as waterworks men and public officials, we are pledged to protect our consumers and to serve them with water which is not only safe, but which is made appetizing to thinking people by a history of clean antecedents all the way from cloud to kitchen tap. Can we afford to ignore this obligation in order to gratify the selfish instincts of a small proportion of the population? The demand to open our lakes does not proceed from a real necessity which must be satisfied—rather it is a luxury, to be indulged as becomes a luxury, certainly not at the expense of others.

We are told by those who advocate recreational use that fees can be imposed for the privileges granted, and the money so raised employed in police measures to prevent pollution of the water. Insofar as the majority of hunters and fishers are concerned the writer likes to believe such measures would be completely effective, but he has had too much experience with self-styled sportsmen to think we can depend upon patrolmen for the high degree of protection we require, and this statement is not to be considered in any degree as impeaching the integrity of these guardians of public safety.

The Advance of Sanitation.—Sanitary engineers tell us it is not only possible, but practicable, to convert the sewage of a city into pure and safe drinking water. Under the stress of circumstance this is already being done, while such procedure will probably become commonplace in the future, yet the thought of drinking reclaimed sewage is repugnant to most people. They want their drinking water kept clean all the time. It is true that purification methods have been so developed that it is possible to convert a grossly polluted river water into a safe supply for human consumption, but to accomplish it the works must be carefully designed and built and constantly watched by highly intelligent and thoroughly trained operators. Even then we find only too frequently that an outbreak of disease occurs through failure of some unit in the process in an unexpected manner. Where only a grossly polluted supply is available reliance must be placed upon purification, but in Southern California, where our raw waters are exceptionally pure, is it not better to maintain their purity than to run the risk of polluting them through human contact?

Recreational Use of Least Importance.—A committee on "Sanitary Conservation of Streams," reporting to the Eighth Annual Conference of State Sanitary Engineers at Chicago, states: "Strenuous efforts must be made to preserve the cleanliness of streams now in that wholesome condition."

The same report, when discussing the order of precedence of use of water resources, states:

"There can be no question that the use of water resources as the source of public water supplies is the first and highest use." Other uses, in order of precedence as set forth by this committee are, sewage disposal, navigation, manufacturing, industry, power, irrigation, agriculture, fishing, or recreation, as may be dictated by local circumstances. Recreational use seems to be regarded as of least importance. Is it wise to wilfully and knowingly run the chance of adding pollution of the most dangerous type to our water just because we think we can take it out? The writer does not believe such practice can be justified by any argument.

The Aesthetic Side of the Question.—Behind the questions of technology, and perhaps of greater effect upon the public mind, is what may be described as the aesthetic reputation of our water. Most people are fastidious in matters pertaining to food, and the present world-wide campaign to popularize knowledge in sanitary and hygienic matters is making them more exacting in this respect. Can we afford to ignore the demand of particular people that the public water supply shall not only be safe, but surrounded by such a clean and inviting environment as to create the belief that it is wholesome and appetizing.

Los Angeles Prohibits Recreational Use of Reservoirs.—For the last eight years Los Angeles has prohibited recreational use of her reservoirs, and we believe the results attained prove the wisdom of our action. Filtration is not practiced, reliance being placed upon natural purification during storage, supported by chlorination. Of course, natural purification in an impounding reservoir is not compatible with recreational use, for it demands at least thirty days' storage amid surroundings where re-pollution cannot take place. We depend upon and benefit through such natural processes to some degree, even in our terminal reservoirs, all of which store water for significant periods, and we cannot afford to sacrifice this advantage, which is really our margin of safety. Without filtration we have an enviable record for a safe water supply—not a single case of typhoid has been ascribed to our water since it has been municipally owned, a period of 27 years. Would our record have been as good had we permitted recreational use of our lakes? Perhaps, but the chance is great that the story would have had a different ending.

The Situation at Los Angeles.—The proposal is frequently made that remote reservoirs might be thrown open to public use, leaving those nearer the point of consumption completely protected. Under some circumstances it might be proper to adopt such a policy, but the Los Angeles system does not lend itself to such a compromise. Our Haiwee reservoir is 200 miles away from the city, in the midst of a practically desert region, but it is, biologically speaking, the key to our system. For more than a hundred miles to the north and west the Sierras send their creeks to feed Haiwee, and natural purification processes in this unique reservoir are depended upon to remove all pollution which may have occurred upon the catchment area, 3,300 square miles in extent. This immense drainage basin is one of California's greatest playgrounds, and no restriction of its use is required or contemplated. The public may hunt, fish, camp and otherwise enjoy this wonderland to the utmost, and Haiwee reservoir will function as a natural barrier to prevent the ill effects of intimate human association with the lakes and creeks from going beyond. But this will be true only if we prevent all pollution of the water in the reservoir. There must be an unbroken period of not less than 30 days during which natural purification can proceed to completion, hence we feel we must also deny recreational use even on our most remote lakes.

During the course of talks to clubs and schools in Los Angeles the writer has frequently put the question as to whether those present preferred recreational use of our reservoirs, and the response has always been overwhelmingly in favor of prohibition. Frankly, I believe this to be a true reflection of public preference in the matter and it has, at least in part, influenced our policy of restriction. Certainly those officials who are held personally responsible for the protection of over a

million and a quarter of human lives against the ravages of water-borne disease are not anxious to wilfully incur any risk, but must always err, if they err at all, upon the side of safety. We must be too strict rather than too lax, and it seems to me that we can conscientiously do nothing less than prohibit recreational use.

Acknowledgment.—The above is an abstract of a paper presented Oct. 25 at 10th annual convention of the California Section of the American Water Works Association.

Meter Testing and Adjusting Bills

By V. E. PERRY

Manager Water Sales Department, Spring Valley Water Co., San Francisco, Calif.

It has been the custom of the Spring Valley Water Co. ever since September, 1918, when San Francisco became 100 per cent metered, to test meters upon the request of a consumer who has a large bill and for which we have been unable to locate the cause.

Let us take, for example, a consumer who has had an increase of about 25 per cent or more in his bill and has received our usual letter (see Fig. 1) calling his attention to the increase and offering to make an examination if the excess has not been accounted for. If, after a careful examination of all the plumbing fixtures and check-up of the house piping, we are unable to account for the increase, we then order the meter tested. We use either of two methods of tests—one is an official test in which the city light and water inspector supervises the test, and the other is an unofficial test. The only difference in these tests is that the light and water inspector is present only in the official test. The test consists of measuring the water through the meter into a tank holding 10 cu. ft. Should a meter test within 2 per cent of 100 per cent efficient, we do not change the instrument, but for all meters that are either fast or slow 2 per cent or more we order a change for a new shop tested meter. As a rule, no increases are made for slow test meters. Our rules governing meter tests, copy of which has been accepted for filing by the Railroad Commission of the State of California, are as follows:

Rule 2. "If a consumer questions the accuracy of a meter supplying his premises, the Company will, upon the request of the consumer, cause an official test to be made at its own expense, provided the meter had not been tested within the period twelve (12) months prior to the date of said request. The consumer shall be notified of the time and place of such test, and may be present if he so desires.

(a) "In case the meter is found to be registering more than two per cent in excess of the actual quantity of water passing through it, another meter shall be substituted therefor, and the Company shall refund to the consumer the overcharge, based upon the correct meter readings, for a period of not more than twelve (12) months last past, unless it can be shown that the error was due to some cause, the date of which can be fixed. In the latter case the overcharge shall be computed back to, and not beyond such time.

(b) "Should more than one test during the year be demanded by a consumer, he shall deposit one (1) dollar with the Company and the test be made as provided in the preceding paragraph. Should the meter be found to register not more than two per cent in excess of the actual quantity of water passing through it, the deposit shall be retained by the Company. Should the meter be found to be registering more than two per cent in excess of the actual quantity of water passing through it the deposit shall be returned to the consumer and adjustment made as provided in the preceding paragraph."

We go a step further than the rules call for by making our adjustments for all meters that are 2 per cent or more fast. Our rules state "more than 2 per cent fast."

During 1928 out of 102,413 active metered services we tested only 494, or about ½ of 1 per cent. Of these 494 meters tested, 217, or 43.9 per cent, were found to be 100 per cent efficient; 193, or 39.1 per cent, were found to be slow; and only 84, or 17.0 per cent were fast. Of the 84 fast meters, 42, or 50 per cent, were 1 per cent fast; 21, or 25 per cent, were 2 per cent fast; 16, or 19 per cent, were 3 per cent fast; and 5, or 6 per cent, were 4 per cent fast.

In accordance with our rules, we refunded for the above fast meters the sum of $71 as follows: for 2 per cent fast $22.18, for 3 per cent fast $31.91, for 4 per cent fast $16.91.

As our gross revenue for 1928 was $6,565,450, our refunds account of fast meters of $71 were only about 1/1000 of 1 per cent of our 1928 revenue. This we believe to be a very favorable showing.

Acknowledgment.—The above paper was presented Oct. 24 at the 10th annual convention of the California Section of the American Water Works Association.

Recarbonation Equipment Installed at Manhattan, Kan.—With a view to facilitating softening and eliminating troubles due to after-precipitation, the city of Manhattan, Kan., has recently installed recarbonation equipment, the first in the state, as an adjunct to its water-softening plant. The equipment burns natural gas to produce the carbon dioxide.

SPRING VALLEY
WATER COMPANY

425 MASON STREET

WATER SALES DEPARTMENT
TELEPHONE PROSPECT 7000

SAN FRANCISCO.

We wish to report a noticeable increase in registered delivery of water for your account, as follows:-

Location:

This month's delivery:

Previous month's delivery:

Increase:

Unless you have reason to believe that the water was beneficially used, we would recommend immediate examination of the plumbing with a view to locating the cause for this large consumption and the repair of any leaky fixtures which may be found.

Very truly yours,

SPRING VALLEY WATER COMPANY

Telephone or write to Adjusting Counter

"Rust, corrosion, the hardening of rubber valves and other changes result in leakage from plumbing fixtures. Small leaks running constantly make little impression on people who do not realize their significance. Yet a leaky toilet may waste without attracting attention, as much water as would supply twenty families."

—From "METER RATES FOR WATER WORKS" by Allen Hazen.

Fig. 1—Letter Notifying Water Consumers of Unusual Increase in Water Consumption

Safeguarding Public Water Supplies

How Weather Conditions Affect Quality and Safety of Supplies

By ARTHUR E. GORMAN
Wallace & Tiernan Co., Newark, N. J.

DURING the last few weeks many New England cities and towns have sought permission from their respective state boards of health to use water from secondary and possibly doubtful sources due to shortage in their regular supply caused by prolonged drouth. Still how many of the officials in charge of these waterworks had kept sufficient data relative to deficiency in rainfall on their watersheds to enable them to predict this shortage and the approximate time the emergency would have to be depended on? And what is more—in how many instances had definite plans been made in advance as to the procedure to be followed when forced to use the emergency supply?

Planning for Emergencies.—A shortage of water for a public supply is a potential health hazard. Every water works system has its one or more weak points regardless of how well it was designed, built and is operated. It is encumbent on the directing waterworks officials to know these weak links when tested by every strain which may be brought on them by those elements called "weather" or any combination of them. He must know his factors of safety under normal conditions, under the extreme conditions which occur seasonally each year and under the most extreme conditions which may be expected every five years and every decade. Not only should he know this, but he should have developed a plan of action to be followed out during emergencies. A plan not housed in a vague way in his own mind but a plan which has been discussed carefully with his assistants and still better practised and put in writing for ready reference.

The average water works operator properly trained will carry on satisfactory under normal conditions and probably do reasonably well under somewhat unusual circumstances. But, these are not the conditions which precede outbreaks of typhoid fever and other water borne diseases. Sharp outbreaks or epidemics come as the result of a collapse in the fabric supposed to exist for public safety. They usually follow a simultaneous combination of circumstances which in accordance with the laws of probability will not be repeated in many years. It therefore behooves the careful water works officials always to be on the alert for emergencies.

One Emergency Problem.—Let us take up one emergency problem and some of its possibilities.

As practical water works men what would happen at your plants if it should rain at an excessive rate of precipitation for say three consecutive hours? Would your dam hold? Would flooding of sewers or sewage polluted streams affect your reservoir, clear well or tunnel shafts? Would water short circuit through your reservoir and cut your theoretical storage of forty days down to forty hours, thereby losing your factor or safety due to storage? Would your supply become turbid, necessitating the use of chemical coagulants? These are all possibilities at the more common type of water works in New England.

Chlorination Control System at Chicago.—During the four years that the writer was charged with the safety of Chicago's water supply, a control system for chlorination was developed which was based largely on a program of "watching the weather." Chicago's intakes in Lake Michigan are fixed. The sources of pollution of the lake are known. Therefore from a control standpoint it became necessary to learn all we could about those factors which would bring this pollution to our intakes. Our studies involved rates of rainfall and the effect of run off into the lake, the time and probable duration of river reversals, the effect of dispersion, the rate of drift of pollution in the lake and the influence of prolonged winds and barometric pressures on the lake levels. Automatic recording rain gauges at four places in the city made it possible to predict—even while the rain was falling—the probable affect the run off would have on the direction of flow of the rivers. Flow vanes at key points in the river equipped with alarm lights and bells gave warning of reversals; recording gauges showed river elevations while automatic recording instruments for wind direction and velocity told the story of what might be expected in the way of drift of pollution toward the intakes once it entered Lake Michigan. In other words, we studied our weak points, planned our campaign for control so that it might be put into operation while the emergency was brewing. Without such a system there would have been four or five occasions each year when we had no real factor of safety and would have operated on a wide margin of chance with the health of 3,000,000 as the stakes.

In the writer's opinion any unusual change in weather conditions, such as—excessive rainfall, high winds, prolonged drouth, excessive heat or cold—warrants the application of emergency operation pending investigation. No harm can come from playing safe. In Chicago with a predicted southerly wind the orders for increase in the chlorine to be used for sterilizing water from intakes exposed to potential pollution were always given. During the periods of rainfall the chlorine dosage was always raised at least ½ to 1 lb. per million gallons at all pumping stations as a safety factor against possible pollution of water in tunnels from sewers located near land shafts. Whenever emergency orders were issued they were not withdrawn until the results of bacterial examination of special samples collected during the emergency had indicated it was safe to do so.

Checking Special Measures Important.—From an operating standpoint it is highly essential that every water plant keep regular routine records of operation. But for protection it is of the utmost importance that whatever special measures are taken during emergency periods be checked for results obtained. Special raw water samples at the height of an emergency, supported by equivalent samples of the water delivered to consumers during the emergency and while your precautionary program is functioning, are of inestimable value. They not only tell you the story of the extent of your emergency by means of which you may be guided in the future should similar circumstances develop, but they serve as your protection in case of controversy resulting from the emergency itself. At Chicago we prepared for these emergency samples by storing special sterile bottles at all sampling points and having emergency sampling equipment on hand at the laboratory for immediate use at any time.

Weather Conditions Should Be Studied.—It is obvious that the smaller water works cannot afford to have the expensive equipment considered necessary at large cities. But still there is no reason why the officials of smaller water works cannot apply fundamental precautions indicated as desirable through a keen study of weather conditions and their relation to the peculiar conditions at their respective plants. Make it a point to study the affect and influence of every unusual period of weather on the operation of your plant and the quality of the water delivered therefrom. Watch for the unusual with that unceasing instinct which makes the experienced Yankee sailor put into port with the first threat of a so' easter.

The water works official who is always on the alert for the unusual rarely has to meet with serious emergencies. He instills into his men that element of watchfulness and carefulness which prepares them to meet the more than average emergency with calm. He steels them to a degree of efficiency and discipline which makes it possible for him to get prompt and effective response to his directions when the real emergencies arise.

Emergencies of any kind in supplying safe water to consumers have a distinct public health aspect. For this reason it is the part of wisdom for the water works officials to take into their fullest confidence the health officers charged with supervision over public water supplies. The weak points in any water works should be known to the health officer. His advice should be sought in establishing a control program. He should be notified of emergencies as they develop or threaten. Your experiences reported to him may save the day for some other water works at a later period; or conversely a warning from him based on a report from some other operator may be just in time to permit you to take the proper control measures. An outstanding example of this is the cooperative program developed through the Ohio River Interstate Stream Pollution Agreement for notification through the state departments of health of water works operators of spills of phenol wastes on the Ohio river watershed.

In conclusion, this paper is a plea to the water works officials to "watch the weather"—that they may know its relations to the weak features of their respective plants; that they may work out an effective plan of action when emergencies arise; and that they may work in full harmony and cooperation with the proper health authorities in producing for the consuming public at all times, clean, safe, wholesome water.

Acknowledgment.—The foregoing is an abstract of a paper presented Sept. 20 at the 48th annual convention of the New England Water Works Association.

Leakage and Wastage at Cambridge, Mass.—The report of the Water Board of Cambridge, Mass., for the year ending March 31, 1929, contains the following:

In 1919, a pitometer survey was made on the system. The City was divided into twenty-two sections and a careful inspection was instituted for leakage and wastage. This year the city was again surveyed, and in seventeen sections the consumption was practically the same as noted in 1919. But in five sections the consumption had increased from 50 per cent to 100 per cent. This increase may be traceable to the additional population due to the construction of apartment houses, of from 20 to 30 suites, which replace the demolished old houses formerly containing one or two families.

Electrically Illuminated Fountain for Filtration Plant

An electrically illuminated fountain, having a number of unusual features, has been installed recently in the park at the filtration plant at Peterborough, Ontario. The installation was described by R. L. Dobbin, General Manager of the Utilities Commission of Petersborough in a recent issue of the Contract Record, from which the notes following are taken.

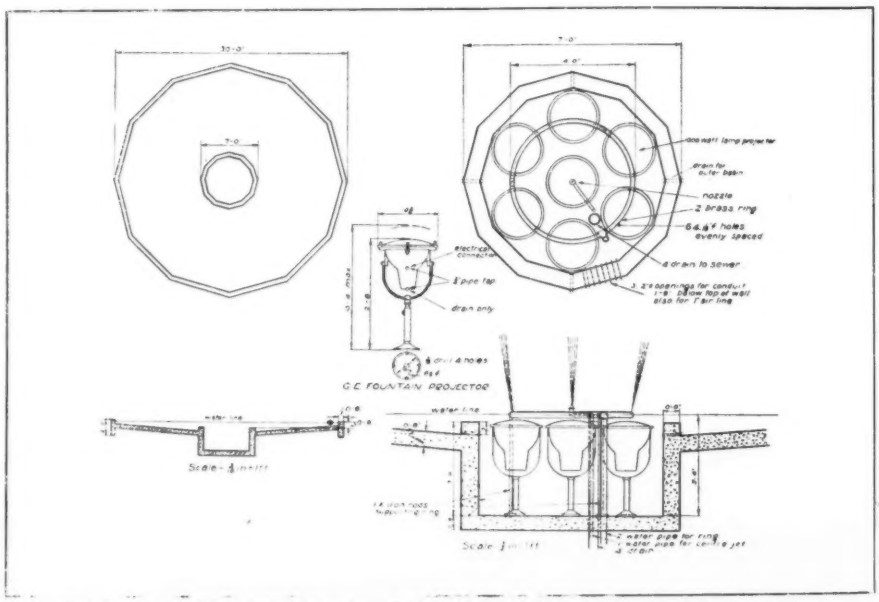
The fountain was designed and installed by the staff of the Petersborough Utilities Commission. It is designed along somewhat unusual lines specially for night illumination purposes.

The fountain itself consists of a shallow pool, dodecagonal in shape, with a center well which houses the lighting projectors and water jets. This is indicated in the detailed drawing. When the fountain is not operating, the only thing visible in the 30-ft. pool is the center nozzle and the top of the 4-ft. spray ring which is half submerged in the center. When in operation the spraying apparatus, of course, is not visible.

The seven projectors, supplied by Canadian General Electric Co., are of the submerged type, each containing 1,000 watt lamps. The one in the center, immediately under the central nozzle, contains a clear lens and the six others arranged on the points of a hexagon (in alternate corners of the dodecagonal wall) under the spray ring are red, green and amber—two of each color.

Current at 110 volts, a.c., is supplied to the projectors through underground lead cables from a flasher that is located in a manhole at a short distance from the fountain. With this arrangement, various combinations of colors are obtained and the flasher may be adjusted while the fountain is in operation. The central jet of water attains a height of 35 ft. and the outer 4-ft. ring of fine spray a height of 10 ft.

During the summer the ever-changing colors against a background of green trees and shrubs have attracted a host of visitors to the park and filtration plant.



Details of the Electrically Illuminated Fountain of the Peterborough Filtration Plant

State and Municipal Bond Election Results During September

New state and municipal bond authorizations reported during September broke all previous records for this year, according to The Daily Bond Buyer, of New York, whose regular compilation shows authorizations by 83 states and municipalities totaling $84,308,700. This compares with $16,870,199 in September last year, $13,372,949 in September, 1927, and $36,164,949 in 1926.

The unusually large total of authorizations last month is largely accounted for in the $55,000,000 program of the city of Philadelphia and $16,200,000 for the state of Maine. Other large items were $3,500,000 Hidalgo and Cameron Counties, Texas, Water Control and Imp. Dist. No. 9, and $1,000,000 Spartanburg, S. C., Sanitary Dist.

Issues defeated amounted to $10,182,600, as compared with $9,707,500 in September last year, $12,292,250 in September, 1927, and $4,178,500 in 1926. Among the larger projects defeated by the taxpayers of 34 communities were $4,070,000, Jefferson County, Texas; $3,000,000, Tacoma, Wash.; $800,000, San Augustine County, Texas, and $453,000, Falls County, Texas, Road Dist. No. 4.

A 22.4-Year Cycle in Rainfall and Its Cause

Analysis Employs Tree Growths, Clay Varves and Sunspot Records Guided by Electromagnetic Whirl Theory

By HALBERT P. GILLETTE

THE accompanying chart shows a rainfall cycle of about double the so-called 11-yr. sunspot period. It is particularly interesting in that the supernormal rainfall of one 11-yr. period is almost exactly offset by the subnormal rainfall of the succeeding 11-yr. period. This chart was recently sent to me by Mr. Burt Harmon, Hydraulic Engineer of the Water Department of Long Beach. I have inserted the dates of the Jupiter sunspot cycle, showing both its date of maximum and minimum effect. It will be noted that Mr. Harmon's 11-yr. periods should be shifted about a year to the right to correspond with the Jupiter sunspot dates; but that doing so does not appreciably alter his calculations of excess and deficient rainfall for each of the 11-yr. periods.

In an article published in "Municipal News and Water Works," October, 1928, I had shown that it is sunspots of south polarity that cause barometric lows, and that it is their preponderance over sunspots of north polarity that largely determines the amount of rainfall. Coupling this fact with Hale's discovery that the leading sunspots have opposite polarities in the sun's two hemispheres and reverse their polarity about every 11.2 years at about the time of sunspot minimum, I had indicated the reason for a 22.4-yr. rainfall cycle; but none the less I was surprised by the striking character of Mr. Harmon's chart. I decided, therefore, to test my theory more fully by taking the data of radial growth of pine trees for 500 years, as given by Douglass, checking the quantitative facts derivable therefrom against my theory. Table I shows the result of this analysis, which yields an almost perfect check of theory against fact for 43 out of 45 cycles of 11 years each. Had all 45 cycles checked the theory as did the 43, such an occurence could happen by mere chance only once in 35 trillion times. However, taking only the 30 cycles subsequent to 1570, in which not one departs from the theory, the probability ratio is ½ raised to the 30th power, making the odds about one billion to one that this coincidence is not fortuitous. Since the theory is thus shown to be in accord with the facts, by odds that are overwhelmingly against the accordance being accidental, it may be regarded as being established as true in so far as it explains the phenomenon in question.

Briefly reviewed, the theory of this phenomenon is as follows: The sunspots, as Hale has proved, are electromagnets, often of tremendous power. Any magnet causes free electrons to rotate around the magnetic lines of force, as laboratory experiments have shown. Electrons thus rotating themselves constitute an electromagnet, as is clearly seen in the case of solenoids. The earth itself is an electromagnet and so is the air above it, for both contain free electrons that are carried around by the earth as it rotates on its axis. Hence, these two magnetic fields must give rotation to free electrons in the air, that is, they must generate cyclonic whirls of electrons in the air. But such whirls in turn are electromagnets and tend to migrate under the action of the magnetic fields. The fields that may effect such migration are numerous, including certainly the earth's two great fields, the sun's great field, the fields of the sunspots and the fields of the planets.

The fact that cyclones rotate in opposite directions in the earth's two hemispheres is in accordance with the theory that the two magnetic poles of the earth would produce opposite rotations of the cyclones. If an ascending cyclonic whirl of air bends backward, it will tend to descend as an anticyclone. The alternation of lows and highs where storms travel across a continent and several other facts bear out this influence. Sunspots commonly occur in pairs that rotate in opposite directions, showing close analogy with our cyclones and anticyclones. But whereas our cyclones never reverse their polarity (or direction of rotation), sunspots reverse it about every 11.2 years.

The magnetic north pole of the sun lags behind the rotation of the photosphere, or moves relatively westward, as Hale has shown. Let this also be the case with the planets and let their magnetic fields be sufficiently powerful, and it follows that they must produce magnetic disturbances in the sun's atmosphere, by causing changes in the density of the free electrons in different parts of that atmosphere. These changes must be most pronounced when the magnetic axis of a planet points most nearly toward the sun, which will be oftener than the orbital period of the planet. This gives an explanation of the difference between Jupiter's orbital period of 11.86 years and the great sunspot period of 11.2 years.

I have found that Uranus produces a sunspot cycle of 81.15 years (its orbital period being 84 years), its last maximum being July, 1870. Uranus is far more powerful than Jupiter in its sunspot effect, probably because its rotative axis is so greatly inclined that its magnetic axis points much more directly at the sun than does that of Jupiter.

Uranus and Jupiter are two great sunspot generators, the latter being exceedingly important because of its relatively short cycle of 11.2 years. Its last well-established maximum sunspot effect was Aug. 12, 1917, or 1917.6; and I have counted backward from that date to establish the 11-yr. cycles in Table I. The last year of each of the cycles in that table is the year in which a Jupiter sunspot maximum effect occurred. This does not mean that there was invariably a maximum of spots in that year, for Uranus, Neptune and perhaps a trans-Neptunian planet at times submerged Jupiter's effect. It is noteworthy, however, that of the 28 recorded sunspot maxima from 1615 to 1917, 17 occurred within a year of the dates that I have assigned to Jupiter maxima, and that 6 others are fully explained by the Uranus and Neptune dates of maxima, leaving but 5 not explained with great precision.

Neither Jupiter nor Uranus produces a sunspot maximum oftener than once during an orbital revolution, which indicates that only one end of its magnetic axis is effective in causing sunspots.

The total number of spots in the sun's two hemispheres is seldom even approximately equal during any two successive 11-yr. periods. Prior to about 1850 the number of spots in the sun's northern hemisphere exceeded those in the southern hemisphere, but since then the contrary has been the case. If cyclonic storms are more numerous and intense the greater the preponderance of sunspots of south polarity and if the polarity reverses every 11.2 years, it follows that so long as there is a preponderance of sunspots in the sun's southern hemisphere, then there must tend to be alternate 11.2-yr. periods of wet and dry weather, provided only that the spots of the new cycle wax rapidly while those of the old cycle wane rapidly in number. Whether this order of waxing and waning occurs or not may be readily ascertained by comparing the average percentages of spots in the two hemispheres, thus:

Period	Per Cent N. Hemisphere
1874-84	43
1885-95	38
1896-06	48
1907-17	45
1918-28	56

The percentages above given are average annual percentages of spotted area (Greenwich data) deduced from a

table given by Clough in "Monthly Weather Review," July, 1928, p. 262. The alternate increase and decrease in percentage for the 11-yr. periods confirms the inference above made.

On the other hand if we take 11-yr. periods from minimum to minimum (instead of from maximum to maximum) of Jupiter sunspot effect, we get the following results:

Period	Per Cent N. Hemisphere
1880-90	38
1891-01	42
1902-12	42
1913-23	57

For the first three of these periods we have an almost exact equality of spotted area in the two hemispheres. Then follows a sudden marked increase, the duration of which is problematical.

In the light of the results shown in Table I, it is very clear that for many 11-yr. periods, there has been alternate rapid and slow growth of the Arizona pines, just as during the 5 cycles from 1874 to 1928 there was an alternation in the percentage of spotted area.

Study of Table I shows that there have been three times since 1403 when the cycles of wet and dry periods of 11 years reversed themselves, namely, the periods of 1515-25, 1638-48 and 1862-72. This indicates that about these times the percentages of sunspottedness in the sun's hemispheres reversed themselves. The earliest accurate records of percentages of spottedness in the two hemispheres are those of Carrington and Spoerer, which are given in Clough's article above mentioned, from 1854 to 1890. For the year 1854 there were 61 per cent of the total number of spots in the sun's northern hemisphere; in 1855 it had dropped to 49 per cent; in 1856 to 37 per cent and from 1856 to 1873, inclusive, it averaged 47 per cent. The data subsequent thereto, but using areas instead of numbers of spots, are given above. It appears, therefore, that about 1856 the spottedness of the sun became greater in its southern hemisphere; and it is to be noted that the 11-yr. cycle that followed this date is marked in Table I as one in which a reversal of cycle occurred. So the only quantitative record of change in relative spottedness in the two hemispheres agrees with the evidence of such a change as derived from the growth of the Arizona trees.

The date of the next reversal of cycle is problematical, for Table I shows two different lengths of time between reversals, namely, 123 years and 224 years.

The cause of the shifting of the majority of sunspots from one hemisphere to the other seems to be the positions of the slower-moving planets relative to the sun's equator.

Huntington (pp. 211 and 216) states that Stewart, Maunder and Pocock independently found that sunspots are either more numerous or larger on the sun's face that is more remote from Mercury, Venus and Earth. In his "Astronomy" (p. 35) Chambers says that Stewart (in his investigation of the effect of Venus) found that spots tend to break out on the face of the sun nearest Venus, but attain their maximum when they are on the face farthest from Venus. It follows, therefore, that a slow-moving planet having an orbit considerably inclined to the sun's equator would probably cause more sunspots north of the equator during one-half of its orbital period, and the converse during the second half. I am inclined to think that one or more unseen planets beyond Neptune are mainly responsible for the greater changes in the sunspot preponderance in the sun's two hemispheres.

If I have spoken with a good deal of assurance about the length and constancy of the Jupiter sunspot cycle, it is not merely because the data in Table I very strongly support such an inference.

Long before this table was prepared, I had found just as convincing evidence

Table I.—45 Periods of 11 Years' Growth of Arizona Pines Showing Alternating Cycles

11-Year Period	Average Yearly Growth m. m.	Theoretical Relative Rainfall	
1403-13	1.94	Wet	(Uranus peak, year 1384)
1414-24	1.40	Dry	
1425-35	1.41	Wet	
1436-46	1.05	Dry	
1447-57	1.25	Wet	
1459-69	1.19	Dry	(Uranus peak, year 1465)
1470-80	1.25	Wet	
1481-91	1.04	Dry	
1492-02	1.08	Wet	
1504-14	.90	Dry	
1515-25	.92	Dry	(Reversal of Cycle)
1526-36	1.18	Wet	
1537-47	1.22	Dry	(Uranus peak, year 1546)
1548-58	1.61	Wet	
1560-70	1.56*	Dry*	
1571-81	1.15*	Wet*	
1582-92	1.06	Dry	
1593-03	1.30	Wet	
1604-14	1.22	Dry	
1616-26	1.31	Wet	(Uranus peak, year 1626)
1627-37	0.79	Dry	
1638-48	0.79	Dry	(Reversal of Cycle)
1649-59	0.83	Wet	
1660-70	0.73	Dry	
1672-82	1.00	Wet	
1683-93	.90	Dry	
1794-04	1.04	Wet	
1705-15	1.05	Dry	(Uranus peak, year 1708)
1717-27	1.08	Wet	
1728-38	0.67	Dry	
1739-49	0.77	Wet	
1750-60	0.65	Dry	
1761-71	0.76	Wet	
1773-83	0.54	Dry	
1784-94	0.71	Wet	(Uranus peak, year 1789)
1795-05	0.63	Dry	
1806-16	0.61	Wet	
1817-27	0.49	Dry	
1829-39	0.67	Wet	
1840-50	0.49	Dry	
1851-61	0.70	Wet	(Uranus peak, year 1870)
1862-72	0.71	Wet	(Reversal of Cycle)
1873-83	0.46	Dry	
1885-95	0.43	Wet	
1896-06	0.40	Dry	

*Only 2 of the 45 periods fail to agree with the theory and this is one of the two.

Note: These annual tree-growths are averages deduced from a table given by Douglass for trees at Flagstaff, Arizona, without adjustment for a declining rate of growth with increasing age of trees. In determining the relative rainfall (whether "wet" or "dry"), the contrast should always be made between the 11-yr. periods that are close together; and if it is desired to ascertain whether a given 11-yr. period is "wet" or "dry" compared with the 11-yr. periods before and after, add the average annual rate of tree-growth immediately before the given 11-yr. cycle to the rate of immediate growth following, and divide by 2. If the quotient is a rate of growth in excess of that shown for the given 11-yr. cycle, then that cycle is relatively "dry."

of the stability of the Jupiter sunspot cycle. Take, for example, the fact above mentioned that by assigning a length of 11.2 years to the Jupiter cycle, 17 of the 27 known sunspot maxima from 1615 to 1917 fall within a year of the calculated Jupiter maximum period. Deduct 5 of the 27 known specks because they were caused by Neptune and Uranus, and there are left 22 of which 17 fall within a year of a calculated Jupiter maximum. Since the Jupiter cycle is 11.2 years; if a spot maximum occurs within 1 yr.., on one side or the other of a Jupiter maximum, we have $2 \div 11.2 = 0.18$, which is the probability ratio of its occurrence purely by accident, or about 1 chance in 5. That 17 of 22 such coincidences occurred could have happened purely by chance only once in 88,414,400 times.

Between the sunspot maxima of 1615.5 and 1917.6 there are 27 cycles of 11.19 years. It is admitted that Wolf's determination of the 1615.5 date is approximate. But fortunately there are two very early dates when very large sunspots were seen by the Benedictine monk Adelmus, namely, Mar. 17, 807, and May 28, 840. These dates are 33.195 years apart, or 3 cycles of 11.065 years, which is close to the Jupiter cycle as to leave little doubt that they were both near the times of a Jupiter sunspot peak. From Mar. 17, 807, to Aug. 12, 1917, is 1110.4 years. If 99 cycles intervene, they average 11.216 years; if 100 intervene, they average 11.104 years. So many other facts indicate that the cycle is about 11.2 years that the 11.216 is probably very nearly correct. If we divide the 1077.2 years between May 28, 840, and Aug. 12, 1917, by 96 we get 11.22 years. According to Chambers (p. 44) Hakluyt saw "a great black spotte" Dec. 7 to 16, 1590, or 326.3 years prior to Aug. 12, 1917, which divided by 29 is 11.248 years. It is significant that the Douglass table of Arizona (Flagstaff) pines shows an exceptionally great growth in 1589, only a year earlier than the Hakluyt observation; and the Douglass table of California sequoia growth likewise shows a peak in 1589. This last-named table furnishes many excellent opportunities to test the theory that there is a cycle of about 11.2 years. I will mention only 3 of scores of similar calculations that I have made, using the sequoia growth tables. In 1157 B. C. and again in 1146 B. C. (11 years later), there were extraordinarily great thicknesses of wood deposited in these trees. From 1157 B. C. to 1917 A. D. is 3074 years, or 274 cycles of 11.215 years. An exceptionally great growth occurred in 663 B. C., which is 2580 years prior to 1917 A. D., and 230 cycles make the average 11.217 years.

Table II gives the years of sunspot maxima as far back as sunspot counts have been accurately made. It also gives the dates that I have assigned to Jupiter's maximum sunspot effect, the Jupiter cycle being 11.21 years.

See an article in the October, 1929, issue of "Water Works and Sewerage," on Five Important Weather and Earthquake Cycles.

The table shows that for the last 8 sunspot maxima, there has been an alternation of high and low numbers of spots. During this period of 80 years, Jupiter has not been seriously interfered with by any planet, except when Uranus produced the peak of 1870.5. The previous peak period of Uranus in 1789 and that of Neptune in 1778 seriously interfered with Jupiter effects. In fact neither in 1783 nor in 1794 did Jupiter cause many sunspots, because of the effects of the magnetic fields of Neptune and Uranus upon Jupiter's field. Because their axes lie almost in the planes of their orbits Neptune and Uranus produce very powerful magnetic effects on the sun, Jupiter and earth.

Although Jupiter causes its maximum sunspot effect only once in 11.2 years, when its north magnetic pole points most directly sunward, it usually causes pronounced weather disturbances when its south magnetic pole (as well as its north magnetic pole) points most directly earthward.

I am inclined to think that sunspots change their polarity every 11.2 years, because there are two magnetic fields on the sun (corresponding to the two on the earth) which alternate in their control of the rotation of sunspots. If electrons are carried aloft as a result of the action of the lower field during one 11.2-year period, then the upper field is closer to the most highly electrified intermediate atmosphere of the sun, and during the succeeding 11.2 years the upper field gives rotation in an opposite direction to the sunspots and discharges streams of electrons back toward the lower fields. A similar effect could be secured by the periodic warping of the two fields under the magnetic attraction and repulsion of the sunspots.

If one of the two fields is stronger than the other, it follows that the 11.2-year sunspot peaks would alternate in intensity.

Sunspot Peak Year	Average Number of Sunspots	Date of Juipter's Max. Sunspot Effect	
1706	49	1704.6	
1718	50	1715.8	
1727	90	1727.0	
1739	85	1738.2	
1750	83	1749.4	
1761	85	1760.6	
1778	155	1771.9	(Neptune peak in 1778-4)
		1783.1	
1787	132	1794.3	(Uranus peak in 1789.3)
1804	47	1805.5	
1816	45	1816.7	
1830	71	1827.9	
1837	137	1839.1	
1848	124	1850.3	
1860	95	1861.5	
1870	139	1872.8	(Uranus peak in 1870.5)
1884	63	1884.0	
1893	84	1895.2	
1906	62	1906.4	
1917	100	1917.6	
1928	76	1928.8	
		1940.0	(Neptune peak in 1939.8)
		1951.2	(Uranus peak in 1951.7)

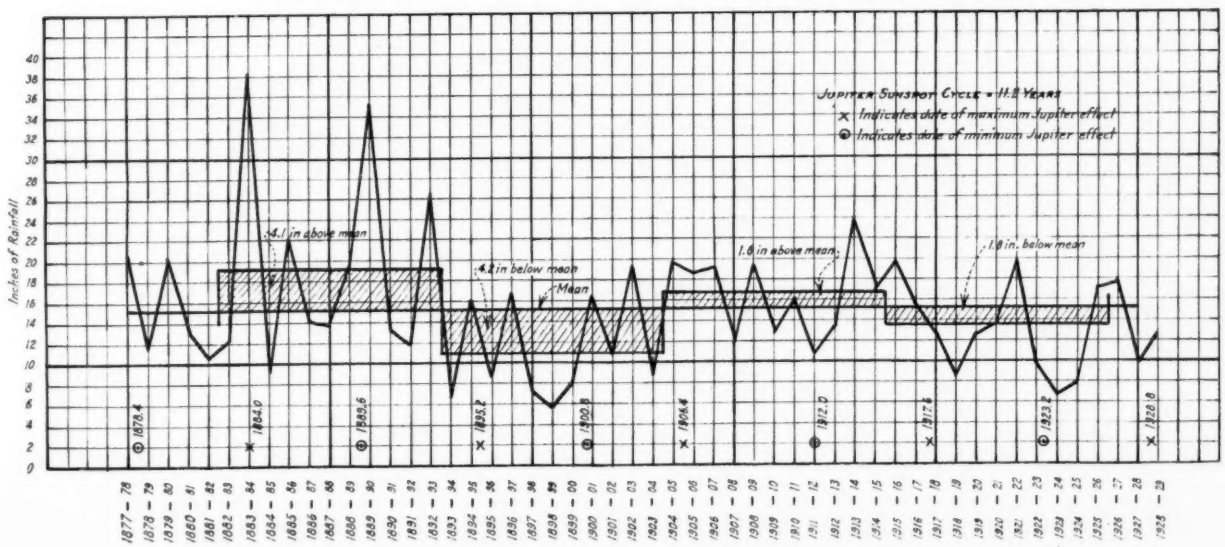
Note: The sunspot numbers prior to 1750 are those given by Fritz (Monthly Weather Review, October, 1928, contains a translation of his paper). The dates of Jupiter's maximum sunspot effect are based upon a sunspot period of 11.21 years deduced by the author. See "Water Works and Sewerage," October, 1929, for a discussion of Jupiter's cycle and those of 4 other planets.

Although the sequoia trees furnish a very long chronology of weather changes, a much longer one is to be found in Baron Gerard de Geer's clay varve charts, given in his great article, "On the Solar Curve as Dating the Ice Age, the New York Moraine and Niagara Falls," "Geografiska Annaler," 1926, H. 4. An English translation of this article was received from Baron de Geer (whose address is Sveavagan 32, Stockholm, Sweden), and its charts have enabled me to calculate the length of the Jupiter sunspot cycle with great precision. Baron de Geer had, of course, seen that there is a frequently recurring cycle of 11 years in the thickness in the clay varves, or layers of clay annually deposited in ancient glacial lakes. The zero year of his varve time scale is about 6800 B. C. From 1077 to 1580 on his time scale is 504 years, and there are peaks in his chart at those dates that are clearly peaks in an 11-yr. cycle; $504 \div 45 = 11.200$ years.

On his chart in the year 6975 of his scale there is a peak that is also clearly one of a series of 11-yr. peaks; hence, from 1077 to 6975 is 5,898 years which divided by 526 gives 11.213 years, and divided by 525 gives 11.234 years. Judging by the sunspot data and the tree-growth data, the proper division is 526. If so, Jupiter's sunspot cycle is very nearly 11.213 years.

This seems to be the acme of accuracy attainable at present until it is remembered that Baron de Geer has stated that he feels very sure that his zero year is within a century of being 6800 B. C. If so, it becomes possible to tie his clay varve chronology to our Christian chronology with accuracy, if we know quite closely two sunspot cycles. As above stated the Uranus spot cycle is very clearly marked both on the sunspot curve and on the tree growth curves. I had found it to be 81.15 years, its last peak being in July, 1870. Study of the clay varve chart indicated Uranus peaks in 1185 and 6784, to name only two extreme years that were clearly defined. Those years are 5,599 years apart, and 69 cycles, give 81.145 years as the Uranus cycle. Using this Uranus cycle and a Jupiter cycle of 11.214 years, I was able to show that de Geer's zero year is 6766 B. C., provided his 6806 B. C. estimate is within 200 years of the correct date.

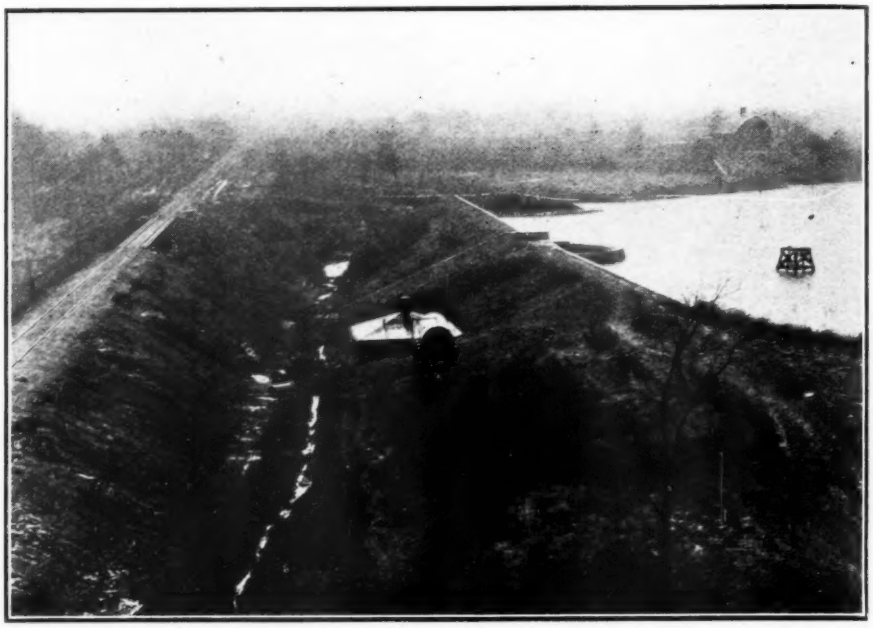
Table I clearly shows the effects of 7 sunspot peaks due to Uranus; the first one in 1384 is foreshadowed in the great growth of 1403-13. The next one, in 1465, finds full expression, for although the decade of 1459-69 shows an average growth below that of the preceding and the following decades, that is because it lies in a dry period. If we take the 3 periods (11-yr.) that center around each of the 6 Uranus peaks of



Rainfall Record for Los Angeles, Calif.
Mean for 51 years, 15.08 in.

1465, 1546, 1626, 1708, and 1789, and 1870, and the 3 periods that center around each of the 6 Uranus valleys of 1425, 1506, 1586, 1668, 1749 and 1830, we find that there was 21 per cent greater tree growth during the 6 Uranus peak epochs (of 33 years each) than during the 6 Uranus valley epochs (of 33 years each); this calculation being based on data of growth equated for declining growth with increasing age, as per the Douglass chart. The six 11-yr. periods nearest a Uranus peak show a 29 per cent greater growth (equated) than do the six periods nearest a Uranus valley.

Douglass found that the tree growth at Prescott correlated with the rainfall at Flagstaff, 12 miles away, in 70 per cent of the year during which rainfall records had been kept. There is thus established a very satisfactory agreement between Arizona tree-growth and rainfall.

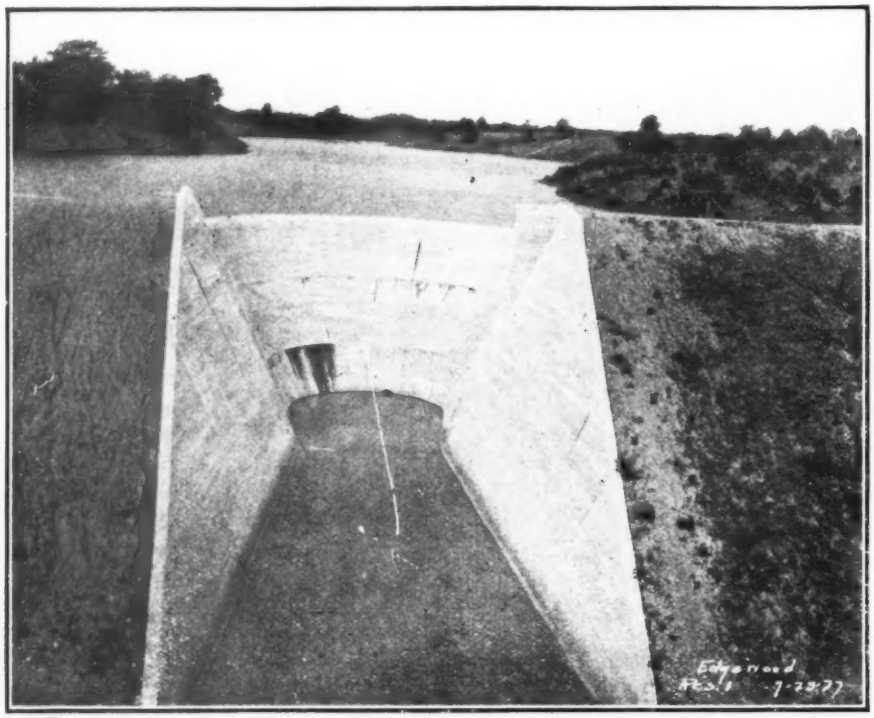


Earth Dam and Spillway, Mango, Ill., Impounding Reservoir on the Edgewood Cut-Off of the Illinois Central

Horseshoe Spillway Solves Problem of Limited Space

Among the many problems incidental to the building of the Edgewood cut-off of the Illinois Central System was the development of a complete water-supply system having a capacity adequate for the needs of future years. The line, which extends from Edgewood, Ill., to Fulton, Ky., has a length of 169 miles and the total annual volume of water required, estimated at 336,000,000 gal., was divided among eight water stations. Aside from the major aspects of the problem of obtaining a suitable supply, many of the details of the project furnished an outlet for the ingenuity of the designers.

One such instance occurred in connection with the Mango, Ill., supply station. Here the water station is supplied from a reservoir having a storage capacity of 109,000,000 gal. and covering an area of 33.8 acres. The total annual runoff draining into the reservoir is estimated at 125,880,000 gal., a quantity which is far in excess of the estimated maximum annual consumption of 40,000,000 gal. The water is impounded by an earth-fill dam, paved with concrete on the reservoir side to prevent erosion. The unusual feature of this dam is the spillway. Space was limited and, in order to secure a length of crest sufficient to care for the excess runoff, the spillway was built in the form of a horseshoe. Dam and spillway are shown in the accompanying illustrations. The spillway has a length of 88 ft. The dam is 523 ft. long and 30 ft. high, and contains 38,300 cu. yd. of earth. A concrete toe wall and a cut-off wall of Wakefield sheet piling are provided at the toe of the fill. The downstream slope is sodded.



Horseshoe Spillway of the Mango Dam. This Design Was Made Necessary by Limitations on the Space Available

The water development along the Edgewood line was under the direction of C. R. Knowles, Superintendent of Water Service for the Illinois Central.

Cleaning Rusty Pipes with Steam

Recent experiments in the removal of rust incrustations from steel pipe with steam are declared to have been successful. The pipe is filled with steam for about 15 minutes and then flushed with cold water, which carries away the loosened scale. These operations are repeated until flushing brings down no more of the oxide, indicating that the inside of the pipe is clean. Steam pressures from atmospheric up to 150 lb. per sq. in. have been used. Since a pipe treated in this manner must be free to expand throughout its length, the process apparently has a somewhat limited application.

This scheme was originated by C. W. Poulson, chief engineer of the Equitable Building in New York City. To date, about 2500 ft. of pipe have been cleaned in lengths varying from 250 to 500 ft.

Long Distance Transportation of Water

An Outline of the Main Factors to Be Considered, Particularly in the Solution of This Phase of the Water Supply Problem in the Semi-Arid Southwest

By E. A. BAYLEY

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MANY important elements must be taken into consideration in making the preliminary investigations for the transportation of large quantities of water over long distances by means of an aqueduct system. Among these, may be mentioned.

1. The determination of the definite point of intake for the aqueduct and its location with relation to any initial storage which may be provided.
2. The location of its terminus and the minimum terminal storage required.
3. The character of the country which the aqueduct is to traverse.
4. The type of aqueduct suitable for the country through which it is to be built, and the material to be used in its construction.
5. The quantity of water to be transported.
6. Whether or not pumping along the route will be necessary.
7. The possibility of generating power along the aqueduct route.
8. The most suitable rates of gradient for its several parts.
9. The estimate of construction costs.

In the case of several routes being available, a comparison of the construction and operating costs of one route with another must be made, in order that the most economical one may be selected.

Location of Point of Intake.—In locating a definite point of intake for an aqueduct, there are two major factors —topographic and hydrographic—which enter into the problem. The topographic factor includes the elevation of the river itself, the possibilities for reservoir sites and the elevations of these sites with respect to the proposed aqueduct. The suitability of the intake for the elimination of silt and other suspended matter such as brush and trees often carried in flood flow, is also a topographic matter of importance.

A proper consideration of the hydrographic features shows that the intake should be placed where there is a certainty of obtaining an ample quantity of water at all times. If taken too far upstream, the requisite supply might not be available due to the small contributing drainage area above. On the other hand, if taken too far downstream, evaporation and seepage losses might increase at a greater rate than the gain from tributaries. This would reduce the stream flow and possibly affect the supply available.

Both the topographic and hydrographic features were given careful attention in locating the point of intake of the Los Angeles Owens River Aqueduct. From a topographic standpoint, the aqueduct had to be so located that water could be discharged into the city's Haiwee Reservoir on the divide between the Owens Lake Basin and the arm of the Mojave Desert known as Indian Wells Valley. From a hydrographic standpoint, it was deemed advisable to place the grade of the aqueduct sufficiently low that a gravity flow from some very large springs, known as Black Rock Springs, could enter the canal. These springs are located about 60 miles beyond the Haiwee Reservoir, and at a slightly higher elevation. With the grade fixed by the relative elevations of Haiwee and Black Rock Springs, the aqueduct location was prolonged northerly to an intersection with the Owens River, where a suitable intake was constructed.

The Colorado River in its lower reaches is situated at an elevation not greatly above sea level. Many cities of the coastal plain of Southern California to be supplied with water from this source, are relatively higher in elevation. Therefore, any intake in this portion of the river would require an aqueduct of a non-gravity type, that is, one through which the water is pumped over at least a part of its length. If a gravity aqueduct from the Colorado River is constructed, the intake would have to be placed far up the river in order to deliver water to the coastal cities.

Storage Requirements. — A liberal amount of terminal reservoir storage is needed when an aqueduct system is operating at its maximum capacity. The storage required by the city of Los Angeles to operate the Owens River Aqueduct system for the year 1928, is given as an example of relative amounts.

During that year, in caring for its domestic demand, approximately 7 per cent of the annual quantity of water transported, was stored during the winter months for use during the following summer. For irrigation in the San Fernando Valley within the city limits, about 40 per cent of the annual quantity used for this purpose had to be stored. In hillside home areas, roughly 17 per cent of the annual demand was likewise taken from storage. During the whole year, the total terminal storage required to provide for the combined domestic and irrigation demands on the Owens Valley Aqueduct system, was approximately 19 per cent of the entire quantity of water transported.

Storage facilities should be located as near the territory to be served as is practicable, and if possible at a higher elevation.

Character of Country to Be Traversed. —In order to determine the nature of the country through which an aqueduct is to pass, it is necessary to have accurate topographic information concerning the territory. If topographic maps of the Geological Survey are available, it is possible to get at once a general idea of the topography of the country to be traversed. If such maps are not yet made, it becomes necessary to make a detailed survey of the area. This topographic survey usually consists of the making of contour maps, showing a contour interval sufficient for the purpose at hand. These can be made either by the transit and stadia method or by the plane table method. In either method, a system of horizontal control must be installed prior to the actual mapping of the ground. If the transit and stadia method is used, the system of control is usually a chained traverse; while if the plane table method is adopted, triangulation is generally best. The plane table method is used by the United States Geological Survey in all of its mapping for the topographic atlas of the United States, and is also the one used on the Colorado River Aqueduct Project. The information gained from the contour maps, supplemented by field examinations, discloses the several kinds of structures of which the aqueduct will have to be constructed. Mountainous areas indicate that tunnels must be used; conduits or canals are the most suitable for level, plain or desert country; and deep canyons require inverted siphons as a part of the system.

In the case of the Colorado River Aqueduct Project, careful studies of the territory involved show that a complex situation of desert, mountain and valley topography exists. Much of the country through which construction work will have to be carried, is vast desert waste land scattered with rugged ranges acting as barriers. There is, however, a stretch of territory near the western end of the proposed aqueduct which is highly cultivated, where orange groves and vineyards abound. This illustrates that there are many complicated problems which must be solved in the lo-

cating of an aqueduct with relation to the varied topographic conditions.

Type of Aqueduct Construction.—Aqueducts have been constructed of stone, wood, brick, concrete and other materials. The most recently constructed aqueducts have been built mainly of concrete. Where valleys intervene, either reinforced concrete or steel inverted siphons have been used. Tunnels are usually lined with concrete, though this lining is sometimes omitted where the rock is exceptionally firm.

The most suitable type of aqueduct for desert country which is subject to cloudbursts, is probably one that is covered, except in the case of inverted siphons, constructed wholly of concrete, and buried throughout its length. This permits torrential surface flows to pass over the aqueduct without damage to its structure. Modern practice varies in the case of inverted siphons of steel or iron. Some have been buried throughout their length, while others have been supported on piers where they may be inspected and repaired from time to time. The latter case is equally satisfactory with respect to cloudburst conditions.

In the case of Catskill Aqueduct for the New York water supply, a type known as "cut and cover" was built. It is constructed of concrete, horseshoe in shape, and only partially buried. The excavated material was used to make the embankment over the structure. While this type of aqueduct is no doubt suitable for Eastern conditions, it is obvious that on our Western deserts, where no definite stream channels exist, and any section of the desert may be subject to extreme erosion from torrential floods, the better practice would be to bury the aqueduct throughout.

Quantity of Water Transported.—In designing an aqueduct to carry an exceptionally large quantity of water, it is necessary to exercise unusual care in determining its correct cross-sectional dimensions. It must have the best hydraulic properties consistent with good constructional design. The construction quantities per unit of length should be a minimum and still obtain an enduring structure. The subject of hydraulic design must be carefully considered in order that the aqueduct when completed and placed in operation will have the required capacity. When properly designed, the flow should closely approximate the theoretical flow indicated by the formulas used.

The amount of water which can be transported in an aqueduct, is dependent upon its velocity. There are various formulas for the computation of the velocity of flow in conduits, canals, tunnels, inverted siphons, flumes, force mains and penstocks such as make up a complete aqueduct system. These formulas are largely based upon experimental data found to exist in other similar structures. Formulas derived by Ganguillet and Kutter, Williams and Hazen, Scobey and others, give satisfactory results. Likewise, good results can be obtained from other exponential formulas derived from the known flows of constructed aqueducts similar to the one proposed.

In the design of the Los Angeles Aqueduct from the Owens River, the Board of Consulting Engineers who reviewed the plans in December, 1906, suggester the use of the Kutter formula with a value of "n"=0.014 for the design of covered concrete conduits, and the Chezy formula with a value of "C" = 90 for riveted steel pipe lines. Since that time, a number of large conduits and pipe lines, notably those of the Catskill and our own Los Angeles Aqueduct, have been built, and numerous measurements of the actual flow in such conduits have been made. Careful examination of these experiments lead to the valuable conclusions as to the best formulas to adopt in the design of any large aqueduct. In applying such formulas, engineers generally use conservative factors. However, the use of factors which are too conservative, in order to insure the rated capacity, may result in far greater cost than necessary.

Necessity for Pumping.—The ideal method of transporting water long distances is by means of a gravity aqueduct, or one which is constructed on a continuous down-grade from its intake to its terminal point. However, in cases where a gravity aqueduct cannot be built within economic limits of cost, pumping must be resorted to, and a non-gravity aqueduct constructed. In such a case, in order to pump the water economically, ample power available at a low cost is indispensable.

The idea of a gravity aqueduct has an appeal to the popular mind, due largely to prejudice brought down from the past. When pumping plants were small and machinery unreliable, there was a lack of dependability in pumping projects. Additional sums were then properly spent to assure a supply by means of gravity, as pumping methods proved unsatisfactory. All this is now changed. This is a mechanical age, and machines have been perfected so they are about the most dependable things in existence. A pump is little more subject to derangement than a tunnel.

Generating Power.—In the building of any aqueduct, gravity or non-gravity, topographic conditions along the route sometimes make feasible the generation of power. The value of power, increasing year by year, justifies a very careful study of the location of an aqueduct to determine this possibility, and may warrant an additional expenditure to obtain it. If the estimated cost of construction, then, is found to be less than the warranted expenditure, the power plants should be built and operated.

In the building of Owens River Aqueduct, the City of Los Angeles took advantage of the difference in elevation between the Fairmont Reservoir and the Lower San Fernando Reservoir for the generation of power. The city has built three large power plants along this section of the aqueduct. These have been operated for a number of years and the net revenue derived from them has more than warranted their construction.

Along the proposed Colorado River Aqueduct, there is also a possibility of generating some return power near its western terminus. Whether this possibility is economically feasible or not depends upon the result of studies now in progress by the Department of Water and Power of this city.

Most Suitable Gradient.—The cost of constructing any aqueduct is largely dependent upon the hydraulic grade used. Any change in gradient will have its effect on the velocity of the water in the conduit. By flattening the slope, the velocity will be reduced and the cross sectional dimensions must be correspondingly increased in order that the carrying capacity will be unaltered. Conversely, by steepening the grade, the velocity will be increased, and the cross-sectional area can be similarly reduced.

When the points of intake and delivery are fixed, it is often possible to obtain the minimum total cost by balancing the gradients between the several types of structures which vary greatly in cost. In this case, the minimum cost is obtained when the gradients are so established that a change in any one affecting its cost will produce an equal and opposite change in the cost of any other.

It is usually considered sufficiently accurate to balance grades graphically. Curves are plotted showing costs in relation to head, for the various sections of the aqueduct under construction. If tangents are drawn to these curves in such a way that the sum of the heads to be consumed is equal to the total available head, the sum of the costs will be a minimum. Dividing the resulting head for each section by its linear length will give the proper gradient for that section. The results are, of course, subject to limitations in that balancing the gradients must not affect the aqueduct location sufficiently to place it in ground more difficult of construction.

In the location of the Los Angeles Owens Valley Aqueduct, the method of balancing grades outlined was used with considerable success. Between Haiwee and Fairmont, a distance of 135 miles, the saving shown between the estimated cost of the aqueduct having balanced grades and that of one without balanced grades, was considerable.

Where an aqueduct is designed with pumping plants along the line, any change in the hydraulic gradient will effect a change in the head pumped against, and consequently will alter the cost of operating the pumps. The cost of pumping is also dependent upon the demand for water by the consumers.

Therefore, in such a non-gravity aqueduct, the cost of pumping must be balanced against interest and other charges on the cost of construction. This balancing of operating costs against interest and other charges for construction must be studied carefully in order to arrive at the most economical combination of grades and lifts. Due to these variable factors, the estimated capital investment in a non-gravity aqueduct may vary within wide limits.

Estimating Construction Costs.—In long distance transportation of water, the product transported is generally delivered to the user at a price which, if computed on the basis of a price per ton, would be but a few cents as against many dollars for all other commodities. In order to accomplish this remarkable feat, the cost of construction to be carefully estimated, and in actual construction, must be kept within reasonable bounds. A difficulty is found in making comparisons, in the lack of finished works of large size, which can be used for guides as to costs. This is particularly so for projects built within the last decade, when labor and material cost have advanced so rapidly. Study must be made of the laws enacted within recent years bearing on compensation and hours of labor, camp sanitation, safety and other similar matters, all of which must be considered in making estimates. The greater use of mechanical equipment offsets to some extent the higher cost of labor, and there must be careful balancing between the greater use of machinery and manual labor. In preparing an estimate, it becomes necessary to analyze each piece of construction and develop costs step by step, taking care that no essential element is omitted. Certain assumptions must be made, and there the designer has to draw upon all available information and decide according to his best judgment as to what plan of operation should be used.

In considering the relative costs of conduit and tunnel, it will be found in ordinary construction for equal capacities, that the cost of tunnel will be about double that of conduit. In general, the items making the total cost may be expressed in terms of labor, material and equipment, freight and hauling, energy and general expenses such as roads and trails, water supply for construction, camp sanitation, hospital and first aid and communication.

Acknowledgment.—The foregoing is an abstract of a paper presented before the Water Supply Section in the Short Course in Public Administration held June 17 to 21 at the University of Southern California.

Household Leakage.—An inspection by employees of the Water Board of Cambridge, Mass., during the past year located the following leaks:

On basins	171
On bath tubs	142
On set tubs	78
On faucets	2,198
On toilets	1,402

Gate Valves and Fire Hydrants

Notes on Their Maintenance and Operation

By CARL A. HECHMER
Department Engineer, Washington Suburban Sanitary District, Hyattsville, Md.

IT IS surprising, especially in small towns and villages, how little attention is given the maintenance of gate valves and fire hydrants on their water systems. The need of good engineering advice and design is recognized and is compulsory in almost every state. Practically every state health department has a law compelling the submission of plans for approval before a water system can be built. The question of good materials is also generally given the utmost consideration and the best valves and hydrants obtainable, in the judgment of those in charge, are purchased and put into the water system. In many instances they are promptly forgotten until an emergency arises and quick, sure operation is necessary. Unless valves and hydrants are given regular attention and properly maintained, the water works superintendent has no assurance that they will function properly. This fact applies to any make of valve or hydrant or to any other mechanical device or piece of machinery. Unfairly the failure reflects on the designing engineer or the manufacturer of the material and the local department is given a clean bill of health.

Valves.—All valves on a water distribution system should be inspected once each year. This is the minimum requirement of the Fire Underwriters' Association. However, semi-annual inspection is advisable, if finances will allow it. Not only will the mechanical condition of the valve be assured, but valve boxes covered by paving material or earth will be located and reset ready for immediate operation in emergency. This is especially true in rural and suburban sections with a large percentage of unpaved streets. When a large leak occurs, excitement prevails and considerable time may be lost locating a valve box. Occasionally a large stone or stones are found down in valve boxes and these conditions are revealed on inspection and removed. In case the valve has to be dug up, uncovering the valve, it is advisable to install new packing in the packing gland which will prevent digging it up for a repacking for a long time. Always repack a valve when it is uncovered for any repairs. Packing is cheap and easily installed while the valve is exposed. To dig up a valve just to stop a packing leak is an expensive operation. Inspection also brings out valves which were carelessly or accidently left closed. Any water distribution system is weakened by gate valves being closed when they should be open.

How a Valve Inspection Should Be Made.—In addition to the box inspection, a valve inspection should proceed as follows: First set the valve key on the valve nut preparatory to turning the stem. Pour a small quantity of kerosene or diluted automobile crankcase drainings down the valve key stem. The oil will run down the key to the operating nut, thence to the valve stem and lubricate the valve stem against the packing gland. Finding its way into the packing gland, the packing will be softened and revived. Then operate the valve up and down several times, bringing the gate down into the groove hard each time and then opening the valve about one-quarter way. Valve troubles, preventing complete shut down, are usually caused by the accumulation of sediment under the seat. Each time the gate is pushed down into the groove a small portion of this sediment is pushed out the sides. The high velocity of the water through the partly opened valve removes what is pushed out and the operation is repeated until the groove is sufficiently cleaned to allow the gate to seat properly. This procedure is especially recommended when valves fail to hold when attempting a shut down. Of course, if the groove is filled with large stones or jointing material, the valve must be dug up and taken apart for repairs. The writer once operated a 16-in. valve up and down thirty times and secured a perfectly tight shut down. The valve had not been operated for a long time, having been declared worn out, since it always allowed a large volume of water to pass with the gate turned down hard with levers on the key.

In operating the valve a check should be made of the number of turns on the stem. Generally for the same size and make of valve the number of turns to close or open the valve completely is the same and a check can be made to see if the stem is bent preventing the valve from completely closing or opening. Either condition should be corrected, as partly open valves weaken the system and, obviously, partly closed valves will not shut off the water.

If the valve stem can be turned around and around with no apparent effect on the gate, the stem is broken and must be repaired.

Geared valves, usually set in manholes, must be given special attention on inspection. The gears should be cleaned with a wire brush and greased with a light clinging grease. The stuffing box should, of course, be examined and by the by-pass valve should receive the same attention as the smaller valves on the distribution system.

Fire Hydrants.—Proper care in the installation of a fire hydrant is necessary and makes maintenance less expensive. Proper drainage of the hydrant barrel after the main valve is closed is essential to prevent freezing of the hydrant in northern climates. The best method is to provide a bed of crushed stone or gravel around the bottom of the hydrant. The amount of stone necessary is dependent on the nature of the soil. In loose sandy soils a much smaller drainage bed can be provided than in a clay soil that absorbs water very slowly. A safe margin is to provide sufficient area to allow the drainage of an amount of water equal to twice the contents of the hydrant barrel. Connections are often made direct from the hydrant drain valve to the sewer. However, there are two serious objections, first the cost and second the direct connection between a sewerage system and a domestic water supply, even though the possibility of contamination reaching the water in the mains is remote. Generally water mains are laid on one side of the street and sewers on the other. Hydrants are installed at the curb on the same side as the water mains to shorten the length of the lead. To connect each hydrant drain across the street to the sewer would prove much more expensive than providing a stone drainage bed.

Fire hydrants must be inspected at least twice each year, in the spring and fall. Four inspections annually should be made, if possible, and this number of inspections is advocated by the Fire Underwriters' Association. In high value districts, weekly inspections are often made during extremely cold weather. Arrangements should be made between the water department, the fire department and any other possible users of hydrants to report to the water department immediately after using a hydrant during freezing weather and an immediate inspection made to insure the good condition of the hydrant. Use of hydrants by private individuals or contractors should be allowed only by special permit and the daily charge made high enough to permit daily inspection during freezing weather. During mild weather the inspection charge can be reduced as weekly inspections are sufficient.

Valuable data may be secured by the fire hydrant inspection by measuring the static pressure, the residual pressure with one nozzle open and the flow of water from the hydrant. Weak points on the distribution system can thus be determined and improvements due to new installation readily noted. System weakness due to unknown closed valves is also discovered and can be corrected.

Procedure for Making a Hydrant Inspection.—The following procedure is suggested for making a hydrant inspection:

First make a physical examination of the hydrant, noting the condition of the operating nut, the nozzle caps and chains and the general appearance of the hydrant. Then listen for leakage with an aquaphone placed on the operating nut. Then with the nozzle caps on tight open the hydrant valve. If static pressure readings are to be taken replace one of the nozzle caps with one connected to a pressure gauge. Note if all the nozzles are tight in the hydrant and whether the drain valve has closed properly. If the drain valve has not closed water will appear around the barrel of the hydrant on the outside. Then close the main hydrant valve and take off one of the nozzle caps. Observe the rate of drainage of the barrel to determine whether any obstruction has entered the drain valve. To clean the drain valve replace the nozzle cap and crack the main valve. As it requires about three full turns open on the main valve to completely close the drain valve on most makes of fire hydrants, the latter is still open on two turns. The full pressure of the water in the hydrant generally will push out any obstruction in the drain hole and the jet effect of the water through the hole will cut away the stone or earth which has packed around the outside. This simple remedy will eliminate the necessity of digging up the hydrant in most cases to correct drainage troubles.

With one nozzle cap off open the hydrant and thoroughly flush out the water in the lead. At this time the observation of residual pressure and rate of flow should be made. Then close down the hydrant slowly. If the hydrant fails to close off entirely, the trouble is probably being caused by an obstruction under the main valve. Do not put extra leverage on the hydrant wrench, but open the hydrant up again and then close it, repeating the operation several times. Do not attempt to close down beyond the point where the obstruction prevents the main valve from seating properly, as the obstruction will become embedded in the main valve and cannot be flushed off, or the valve seat will be damaged. Both valve and valve seat are damaged by attempts to force closing against obstructions. Foreign matter can generally be dislodged by flushing and it is then not necessary to dismantle the hydrant.

After the hydrant is closed and the rate of drainage checked, the stem should be oiled and the nozzles greased with a graphite grease. This kind of grease will not wash off and it prevents the nozzle caps from sticking.

Inspection Records.—Record of the inspection should be kept on a field sheet. The location of the hydrants to be inspected can be entered on the sheet in the office. These will allow correct routing and prevent skipping any hydrants. The information on the field inspection sheets can then be transferred to a permanent record card in the office. A card should be in the file for each hydrant, stating the size, make of hydrant and information regarding pressure at the hydrant. The date of each inspection should be entered on the card together with the observed pressures and rate of flow. Necessary repairs and their costs can also be recorded on the card and a history of each individual hydrant is easily obtained.

For quick reference the following is suggested: Use a map of the water system mounted on wall board with a push pin to denote the location of each hydrant. The color of the pin can show the condition of the hydrant as reported daily by the hydrant inspectors and the repair foreman. Use a blue pin for a hydrant in good condition, a black pin for a hydrant in need of minor repairs but not out of service, and a red pin for a hydrant out of service. Then when an inspector reports a hydrant out of service it is possible to tell at a glance whether the hydrants around it are in good condition. If several makes of hydrants are in use it is an advantage to know the make of hydrant before sending out a repair gang so the proper parts can be carried along. By using a small round colored disc under each push pin it is possible to determine at a glance the make of each hydrant, a different color being used for each make of hydrant.

For the sake of appearance and to protect the metal, hydrants, should be painted once every two years and if possible annually. Use a good grade of paint and care should be exercised to prevent getting paint on the nozzle threads or operating nut. The grooves on the nozzle caps which hold the chain links must be thoroughly brushed out as excess paint will harden and retard the fire department in removing the nozzle caps to connect fire hose.

Acknowledgment—The foregoing is taken from the July Journal of the American Water Works Association.

National Parks Cover 12,113 Miles.—In the last year the area of the national park system has been enlarged to 12,113 square miles, an increase of nearly 200 square miles, through the addition of lands to existing parks and the establishment of new ones, according to the records of the Department of the Interior. The greater portion of the new territory was added through legislation enacted at the second session of the Seventieth Congress. An area of about three square miles of spectacular scenic beauty was recently added to the Acadia National Park on the coast of Maine through the acceptance by the Secretary of the Interior of deeds covering this land.

Financing the Operation of Sewage Disposal Plants

A Discussion of Sewer Service Charge or Sewer Rental Charges Plans

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IT is possible that every plant operator and city official in cities having a sewage disposal plant, is interested in its proper operation, even though at times from outward appearances at least, his interest may seem to be dormant. He, she or they, as the case may be, may be ever so interested, and still be unable to do anything about it. While it is true that there is at present a decidedly awakened "sanitary conscience" on the part of the public generally in connection with municipal sanitation, the fact remains that taken by and large the sewage disposal plants throughout the country hold a position somewhat like that of an unwelcome orphaned child. Some acquaintance with them is necessary, but it is not to be cultivated. The country is literally covered with disposal plants not giving results easily obtained with just a reasonable amount of maintenance and operation. In many cases, the stream or water courses into which the effluents from the plants discharge, is in no better condition than before the plant was installed. The expenditures for sewage disposal plants throughout the country run into many millions of dollars, which means a tremendous economic waste if they are not properly maintained and operated. There is probably no other line of endeavor either public or private where vast sums are expended, and then allowed to go without attention or maintenance, let alone operation. What is the matter with sewage disposal?

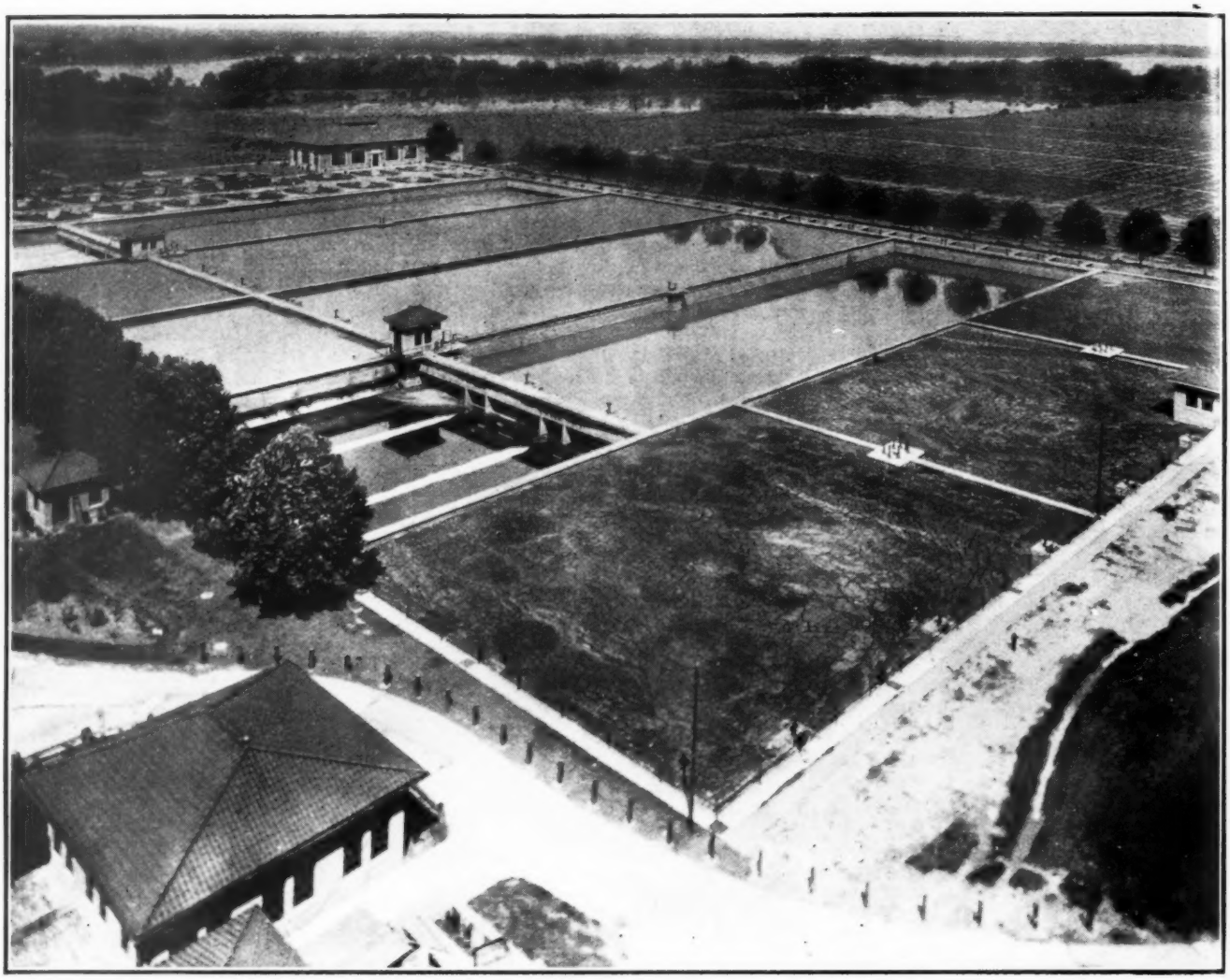
Contributing Causes for Neglect of Sewage Disposal Works.—There are several contributing causes for the general feeling of apathy toward the average disposal plant. Quite a number have been installed under orders from state boards of health, in order to protect water supplies or prevent nuisances. Others have been built on account of threatened or actual damage suits filed by owners over or through whose land the effluent is discharged. In either of these cases the plant is forced upon the city, and is not built on account of any real desire on the city's part to clean up pollution or nuisance caused by its wastes. Such conditions do not create a sympathetic feeling at least on the part of the public at large. It is interesting to note here, that in practically every case where the public interest is aroused, even though perhaps selfishly, excellent operation and maintenance is usually obtained. Reference is made to cities where lack of proper operation would effect those same cities, such as the ones along the Great Lakes, where effluents discharge into waters used for water supply; where sewer outlets are upstream from water intakes; or where effluents flow through the city, etc. Improper design is perhaps another contributing cause for neglect of plants. While sewage disposal is one branch of sanitary science that calls for the highest skill and knowledge to obtain proper design for each problem, and undoubtedly a large number of plants do not reflect engineering excellence, it is doubtful if poor design can be justly used as an excuse for poor or no operation. It is probable that the great majority of plants with improper design, could with technical guidance in operation, be made to function at least satisfactorily. This of course excludes plants that are heavily overloaded, or that have gotten into such a state of repair that operation is impossible.

Lack of Funds Handicap to Proper Operation.—The factors mentioned above as effecting operation of sewage disposal plants adversely are only contributory. There is one outstanding and in fact universal handicap to proper operation, i. e., lack of funds. The plants themselves are built with funds raised by bond issues or by special assessments, but generally speaking no arrangements are made for financing the maintenance and operation. Of course, every city has its "General Fund" to fall back upon, but it is the rule rather than the exception that this fund is already carrying all the burden possible. In some states the amount that can be raised by general taxation is limited by statute, and back of that is the usual desire on the part of the city officials to reduce rather than increase taxes, a very worthy and proper desire when it can be done.

The urban population is increasing steadily and practically and all cities are increasing in size each year. Additional demands are constantly being made for more and better municipal service, such as water, light, garbage removal, sewerage, etc. It is indeed a fortunate group of city officials that can keep taxes from raising, let alone reduce them, assuming of course that extravagance and waste is not practiced. Therefore, it is perhaps natural that the need of funds for operation of sewage disposal plants has been shoved into the background. It is the purpose of this paper to discuss a means of solving this financial embarrassment in which the sewage disposal industry finds itself.

Financing Sewer Rental Charges.—The plan has been styled "Sewer Service Charges" or "Sewer Rental Charges" and is in actual operation in at least two states, namely Ohio and Michigan, and is being considered seriously by health authorities in many of the other states. The plan is made possible by an enactment of the state legislature, authorizing cities having sewerage, sewage disposal plants, etc., or contemplating acquiring or building any or all of such improvements, to levy a service charge against all property connected to the system. In the two states mentioned, the statutes give the governmental authorities great power in assessing and collecting these service charges or rentals. Attention is called here to the two statutes mentioned, copies of which are given elsewhere in this article. The statutes authorize the governing bodies of municipalities to assess a sewer service or rental charge, and gives the power to collect such charges in the same manner as other taxes. The assessment can be levied along with the other taxes if desired. The Michigan statute has been amended several times, the copy given herewith being the present law. It goes somewhat further than the Ohio law, in that it authorizes the issuance of bonds for construction purposes in excess of the amount authorized by statute and includes garbage as well as sewage. In other words, there is no limit for such issues, but the bonds are not a direct obligation on the city. The bonds would be similar in character to the "Utility Bonds" that are authorized in some states. With such bonds, the property mortgaged is the sewers and disposal plant only. The purpose of this law no doubt was to give financing power to cities already bonded to the legal limit, and to small cities where the amount that could be raised under statute would be insufficient. It is questionable whether it would not be better to authorize additional bonding power for sewers and sewage disposal, issuing general obligation bonds, as their market value would be higher than the so-called "Utility Bonds," and thereby give a more economical means of financing. At any rate, quite a number of cities in each of the above mentioned states have enacted ordinances establishing schedules or rental charges, and have or will have funds to properly operate and maintain their disposal plants.

It seems that there can be little question about the desirability of having laws passed in all states, making the charging of sewer rentals possible.



Tanks and Sludge Beds at Back River Plant, Baltimore, Md.

Plants that are now neglected because of lack of funds, could be given proper attention. It is also probable that sewage disposal and general sanitation will receive more consideration, as the importance and need of such improvements will be constantly brought to the attention of the public, by the potent means of a bill for rental. The system of charging for service rendered will put sewage disposal, garbage collection and disposal, and other sanitary measures on the same general plane of respectability as other public utilities.

Rental Details Left to Governing Body.—The Sewer Rental laws in Ohio and Michigan leave the details of the rental charges to the governing body in each particular case, with the exception of privately owned systems in Michigan, which is wise. Rental charges for privately owned systems in Michigan are to be determined by the Public Utility Commission. The question of what rentals to charge is one that will require considerable thought and study in each particular case. Most of the charges that have been established so far are based on the amount of water consumption. This is probably a fair basis in small cities where the sewage is strictly domestic. In larger cities, especially those having a number of industrial plants, the situation is different. Here some industries may have a large water consumption and still contribute a small amount of sewage, also two industries may contribute the same amount of sewage with the sewage from one being four or five times stronger than the other. Also, certain industrial wastes may have a very destructive effect on sewers and operation of the disposal plant due to chemicals. It is possible that the use of water consumption data, tempered by a study of the bio-chemical oxygen demand of the different industrial wastes will give a fair basis for establishing rentals. As stated, the question of proper rentals will require considerable study, as most cities are urging industries to come to them, and great care will have to be exercised to avoid discrimination between the "wet and the dry" industries, although it is probably just that a wet industry contributing a difficult sewage to purify should include an extra item in production costs to cover its disposal. With properly adjusted rates it will be possible to obtain enough revenue to properly operate and maintain any disposal plant, without imposing a hardship on any one. In fact, the rentals required, compared with other utilities, will be in most cases ridiculously small.

Operators or city officials can render a real service to their cities, and to health and sanitation in general by using their influence in the passage of a sewer rental law in their respective states. There should be no legal limits to bonding power of any city for the construction of necessary sanitary works, and the raising of funds for their maintenance and operation.

MICHIGAN STATUTE

STATE OF MICHIGAN, 55TH LEGISLATURE
REGULAR SESSION OF 1929

Senate Bill No. 329 (File No. 350)
Introduced by Senator Woodruff

SENATE ENROLLED ACT No. 99

Ordered to be known as the Dykstra Act

An act to amend the title and sections one, two, three, four, five, six, seven, eight, nine and ten of act number three hundred twenty of the Public acts of nineteen hundred twenty-seven, entitled "An act authorizing cities and villages to provide a sanitary means of disposing of the sewage and night soil thereof; to charge owners or occupants of premises therefor; to raise money and issue bonds to erect and maintain sewage disposal plants, and to repeal act number two of Extra Session of nineteen hundred

twenty-six, and any other acts or parts of acts inconsistent herewith," and to add three new sections to said act to be known as sections twelve, thirteen and fourteen.

The People of the State of Michigan enact:

Section 1. The title and sections one, two, three, four, five, six, seven, eight, nine and ten of act number three hundred twenty of the Public Acts of nineteen hundred twenty-seven, entitled "An act authorizing cities and villages to provide a sanitary means of disposing of the sewage and night soil thereof; to charge owners or occupants of premises therefor; to raise money and issue bonds to erect and maintain sewage disposal plants, and to repeal act number two of Extra Session of nineteen hundred twenty-six, and any other acts or parts of acts inconsistent herewith," are hereby amended, and three new sections are hereby added to said act, said amended title and sections and added sections to read as follows:

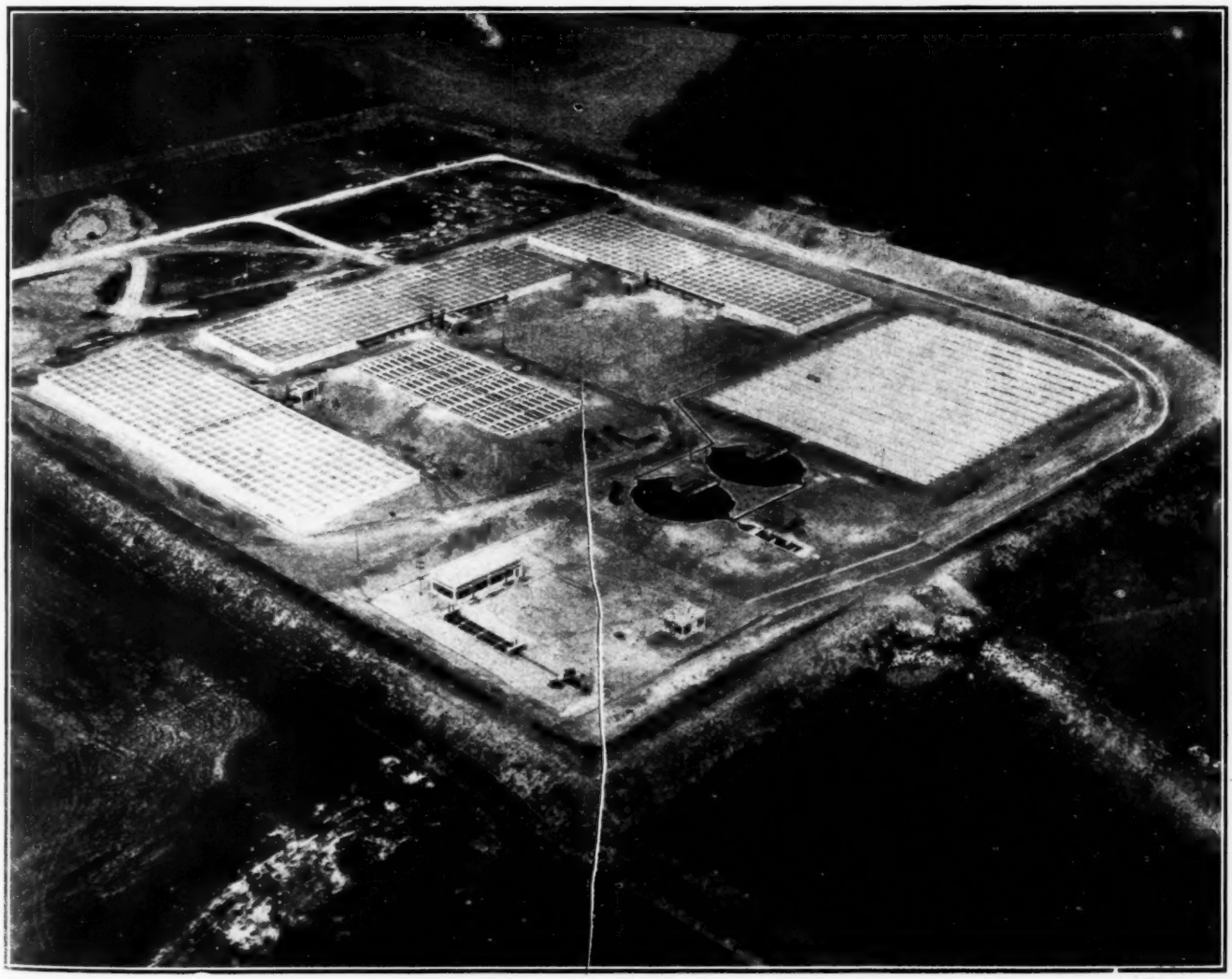
TITLE

An act authorizing counties, cities and villages, either individually or jointly by agreement, to provide a sanitary means of disposing of the garbage, sewage and night soil thereof; to charge owners or occupants of premises therefor; to borrow money and issue bonds to own, acquire, construct, equip, operate and maintain intercepting sewers and sewage disposal plants, and garbage disposal plants, and to repeal act number two of the Extra Session of nineteen hundred twenty-six, and any other acts or parts of acts inconsistent herewith.

Section 1. Any county, city or village in this State, either individually or jointly by agreement, is hereby authorized to own, acquire, construct, equip, operate and maintain either within or without the statutory or corporate limits of such governmental agencies or municipalities, intercepting sewers, including pumping stations, a plant or plants for the treatment, purification and disposal in a sanitary manner, approved by the State Department of Health, of the liquid and solid wastes, sewage and night soil and garbage of such governmental agencies and municipalities. They shall have authority to acquire by gift, grant, purchase or condemnation necessary lands therefor, either within without the statutory or corporate limits of such governmental agencies or municipalities. For the purpose of acquiring property for the uses herein mentioned such governmental agencies or municipalities may invoke and shall have all the rights, powers and privileges granted to "public corporations" under the provisions of act number one hundred forty-nine of the Public Acts of nineteen hundred eleven, being sections three hundred fifty-three to three hundred seventy-three inclusive of the Compiled Laws of nineteen hundred fifteen as amended, and which shall be in addition to any powers granted by such governmental agencies by statute or municipalities by their charters.

Section. 2. Such intercepting sewers, pumping stations, sewage disposal plant and system, and garbage disposal plant and system, shall be considered a public utility within the meaning of any constitutional or statutory provision for the purpose of acquiring, purchasing, owning, operating, constructing, equipping and maintaining such intercepting sewers, pumping stations, sewage disposal plant and system, and garbage disposal plant and system. Governmental agencies or municipalities may issue faith and credit bonds or mortgage bonds therefor beyond the general limits of the bonded indebtedness prescribed by law except as hereinafter provided. Such mortgage bonds as provided in this section shall not impose any general liability upon the governmental agencies or municipalities but shall be secured only on the property and revenues as hereinafter provided of such utility including a franchise, stating the terms upon which, in case of foreclosure, the purchaser may operate the same, which franchise shall in no case extend for a longer period than twenty years from the date of the sale thereof on foreclosure. Such mortgage bonds shall be sold for not less than par, bear interest at a rate not to exceed six per cent per annum, and the total amount thereof shall not exceed sixty per cent of the original cost of the utility except as hereinafter provided. No bonds shall be issued as general obligations of the governmental agencies or municipalities except upon a three-fifths affirmative vote of the qualified electors of such governmental agencies or municipalities and except as hereinafter provided, not in excess of three per cent of the assessed valuation of the real and personal property of such governmental agencies or municipalities as shown by the last preceding tax roll. No bonds shall be issued as faith and credit bonds or mortgaged bonds of the utility except upon a three-fifths affirmative vote of the legislative body of such governmental agency or municipality.

Section 3. The legislative body of any such governmental agency or municipality or the respective legislative bodies of such governmental agencies and municipalities, who may have agreed to jointly own and operate intercepting sewers or sewage treatment plants, may create a separate board or may designate certain officials of the governmental agencies or municipalities, to have the supervision and control of such intercepting sewers or sewage and garbage disposal plants. The legislative body, respective legislative bodies, or such board may make all necessary rules and regulations governing the use, operation, and control thereof. The legislative body or respective legislative bodies may establish just and equitable rates or charges to be paid to them for the use of such disposal plant and system by each person, firm or corporation whose premises are served thereby. If the service charge so established is not paid when due, such sum may be recovered by the governmental agencies or municipalities in an action of assumpsit, or it may be certified to the tax assessor and assessed against the premises served, and collected or returned in the same manner as other county or municipal taxes are certified, assessed, collected and returned.



Aerial Photograph of Sewage Disposal Plant, Fort Worth, Tex.

Section. 4. Bonds which are issued and secured by a mortgage on the utility as hereinbefore provided shall not be a general obligation of the governmental agencies or municipalities, but shall be paid only out of revenues received from the service charges as provided in the preceding section, or from a sale of the property and franchises under a foreclosure of the mortgage. If a service rate is charged, to be paid as herein provided, such portion thereof as may be deemed sufficient shall be set aside as a sinking fund for the payment of the interest on said bonds, and the principal thereof at maturity.

Section 5. Governmental agencies or municipalities as herein mentioned instead of owning and operating a sewer system and sewage disposal plant, garbage collection and disposal system, as herein provided, may grant a franchise for a period not to exceed thirty years to any private corporation organized under, or authorized by the laws of this State to engage in such business, to build, construct, own and operate a sewage or garbage disposal system for the purpose of receiving and treating sewage and night soil and garbage as hereinbefore mentioned from such governmental agencies or municipalities. Such franchise may authorize the corporation to charge each person, firm or corporation owning property from which such sewage or garbage is received such a fee therefor as may be determined to be reasonable by the Public Utilities Commission of this State, upon proper application made either by such corporation, governmental agencies or municipalities and after a hearing thereon. Such franchise may also grant to the corporation the right and privilege to lay all such necessary intercepting and other sewers and connecting pipes in the streets and public alleys of the governmental agencies or of the municipalties as may be necessary to receive and conduct the sewage to the disposal plant and under such reasonable rules, regulations and supervision as may be established by the governmental agencies or the municipalities. Any such franchise shall be void unless approved by three-fifths of the electors of such governmental agencies or municipalities voting thereon at any general or special election.

Section 6. Any such governmental agencies or municipalities may enter into a contract with any corporation organized under or authorized by the laws of this State to engage in the business herein mentioned, to receive and treat in the manner hereinbefore mentioned the sewage, night soil and garbage thereof. Such contract may authorize the corporation to charge the owners of the premises served such service rate therefor as the Public Utilities Commission of this State may determine to be just and reasonable, or the governmental agencies or municipalities may contract to pay the said corporation a flat rate for such service, and pay therefor out of their general fund or assess the owners of the property served a reasonable charge therefor to be collected as hereinbefore provided and paid into a fund to be used to defray such contract charges.

Section 7. Whenever a court of competent jurisdiction in this State shall have ordered the installation of a sewage or garbage disposal system in any of the governmental agencies or municipalities herein mentioned, and the plans therefor shall have been prepared, and approved by the State Commissioner of Health, the legislative body or the respective legislative bodies thereof shall have authority to issue and sell the necessary bonds for the construction and installation thereof, including the disposal plant, and such intercepting and other sewers as may be necessary to permit the effective operation of such system; such bonds to draw interest at not to exceed six per cent per annum, and payable in not to exceed thirty years from the date of issuance; the legislative body or the respective legislative bodies to determine the denomination of said bonds and the date, time and manner of payment. The amount of such bonds either issued or outstanding shall not be included in the amount of bonds which the said governmental agencies or municipalities may be authorized to issue under any statutes of this State or charters. Governmental agencies or municipalities issuing bonds hereunder in excess of the limit of the authorized bonded indebtedness fixed by statutes or charters, the principal and interest of which are not to be paid out of funds created from service charge, as hereinbefore provided, may raise such a sum annually by taxation as the legislative body or respective legislative bodies may deem necessary to pay interest on such bonds, and to create a sinking fund to pay the principal thereof as it falls due. Such annual amount may be in excess of the authorized annual tax rate fixed by the statutes or charters. Except as otherwise provided in this act, all bonds issued hereunder shall be issued and sold in conformity to the provisions of act number two hundred seventy-three of the Public Acts of nineteen hundred twenty-five or as it may hereafter be amended.

Section 8. Whenever an order shall have been made by any court of competent jurisdiction, as hereinbefore provided, the fact that such order was issued shall be recited in the official minutes of the legislative body or the respective legislative bodies. The said body or bodies shall thereupon require that plans and specifications be prepared of such sewage or garbage disposal system, including the necessary storm and sanitary sewers, which plans when submitted and approved by the legislative body or respective legislative bodies shall be submitted thereafter to the State Commissioner of Health for his approval. If such plans are so approved, the legislative body or respective legislative bodies shall thereupon authorize the issuance and sale of the necessary bonds to establish the proposed system.

Section 9. The authority hereby given shall be in addition to and not in derogation of any power existing in any of the counties, cities or villages under any statutory or charter provisions which they may now have or may hereafter adopt.

Section 10. Proceedings under this act shall be taken only in a court of competent jurisdiction in the county in which the proposed interceptors sewage or garbage disposal plants are to be constructed.

Section 12. Whenever it is deemed expedient for the safety and health of the people, governmental agencies and municipalities are hereby authorized to enter into agreement with each other to raise money, issue bonds to erect and maintain intercepting sewers and sewage treatment plants.

Section 13. Whenever governmental agencies and municipalities shall desire to act under the provisions of this law, the relationship established between such governmental agencies and municipalities shall be fixed by contract, and such contracts may be made by governmental agencies and municipalities under the provisions of this act in a manner and to the extent that natural persons might make contracts for like purposes. Such contracts before becoming operative shall be approved by a vote of the majority of the members elect of each of the respective legislative bodies of the governmental agencies and municipalities operating under the provisions of this act.

Secton 14. If any clause, sentence, paragraph, section or part of this act shall for any reason be adjudged or decreed to be invalid by any court of competent jurisdiction, such judgment or decree shall not affect, impair or invalidate the remainder of this act, but shall be confined in its operation to the clause, sentence, paragraph, section or part hereof directly involved in the controversy in which said judgment or decree shall have been rendered.

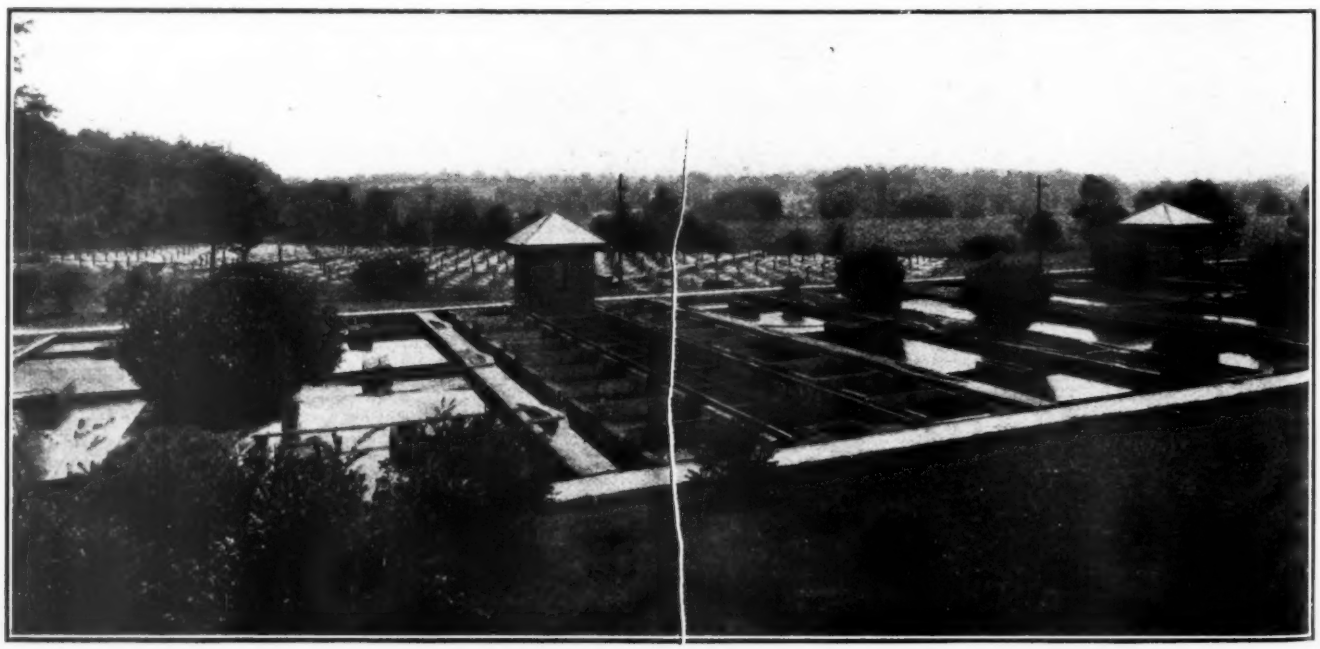
This act is ordered to take immediate effect.

OHIO SEWER RENTAL LAW

(Section numbers, Ohio General Code. Effective July 30, 1923.)

Sec. 3891—1. The council of any city or village which has installed or is installing sewerage, a system of sewerage, sewage pumping works or sewage treatment or disposal works for public use, may by ordinance establish just and equitable rates or charges of rents to be paid to such city or village for the use of such sewerage, a system of sewerage, sewage pumping works or sewage treatment or disposal works by every person, firm or corporation whose premises are served by a connection to such sewerage, system of sewerage, sewage pumping works or sewage treatment or disposal works. Such charges shall constitute a lien upon the property served by such connection and if not paid when due shall be collected in the same manner as other city and village taxes. The council may change such rates or charges from time to time as may be deemed advisable. Provided, however, that in a municipality operating under a municipal charter the council or other legislative body may establish the schedule of rates herein authorized and provide for its administration by designating the department or officer of the municipality to be charged with the enforcement of the provisions of this act. (110 v. 370.)

Sec. 3891—2. In a city the director of public service shall manage, conduct and control the sewerage system and sewage pumping, treatment and disposal works and when the council has established a schedule of rates or charges of rents for their use shall collect sewer rentals, and he shall appoint the necessary officers and agents for such purposes. (110 v. 370.)



Sewage Disposal Plant, Peach Creek, Atlanta, Ga.

Sec. 3891—3. When the council of a village has established a schedule of rates or charges of rents for the use of the sewerage system and sewage pumping, treatment and disposal works, the board of trustees of public affairs shall manage, conduct and control such sewerage system and sewage pumping, treatment and disposal works and shall collect sewer rentals and shall appoint necessary officers and agents for such purposes; and shall assume all duties, in relation to sewers, imposed upon the street commissioner by Section 4364 of the General Code, whereupon the street commissioner shall be relieved of such duties. (110 v. 370.)

Sec. 3891—4. The director of public service and the board of trustees of public affairs may make such by-laws and regulations as may be deemed necessary for the safe, economical and efficient management and protection of the sewerage system and sewage pumping treatment and disposal works; and for the construction and use of house sewers and connections to the sewerage system. Such by-laws and regulations shall have the same force and effect as ordinances when not repugnant thereto or to the constitution or laws of the state. (110 v. 370.)

Sec. 3891—5. The funds received from the collection of sewer rentals shall be deposited weekly with the treasurer of the corporation. Money so deposited shall be kept as a separate and distinct fund and shall be known as the sewer fund. When appropriated by council, it shall be subject to the order of the director of public service of a city or board of trustees of public affairs of a village. Such director or board shall sign all orders drawn on the treasurer of the corporation against such fund. This fund shall be used for the payment of the cost of the management, maintenance, operation and repair of the sewerage system and sewage pumping, treatment and disposal works and any surplus in such fund may be used for the enlargement or replacement of the same and for the payment of the interest on any debt incurred for the construction of such sewerage system and sewage pumping, treatment and disposal works, and for the creation of a sinking fund for the payment of such debt, but shall not be used for the extension of a sewerage system to serve unsewered areas or for any other purpose whatsoever. (110 v. 370.)

Acknowledgment.—The foregoing is a paper presented Oct. 23 at the 5th annual Missouri Water and Sewerage Conference.

Water Filtration Plants in Pennsylvania

Some interesting information on the purification of water in the state of Pennsylvania was given by H. E. Moses, Assistant Chief Engineer, Pennsylvania Department of Health, in a paper presented at the 33rd annual meeting of the Pennsylvania Water Works Association. The notes following are taken from his paper.

The first public water works in the United States were those of Boston, and built in 1652. Eighty-four years later the first Pennsylvania water works was installed at Schaeffertown in 1736. In this state, at Bethlehem, in 1754, there was put into operation the first water works pumping plant in the United States. One hundred and fifty-three years after the installation of the Schaeffertown works, Pennsylvania saw its first filter plant.

Records of the early water filtration plants are meagre, but it appears that in 1889 one was built at New Bethlehem, Clarion County. This was a single unit of the rapid sand type with a daily capacity of 150,000 gal. Three years later the next plant was built at Beaver Falls, Beaver County, replaced by a new plant in 1924, while the Gettysburg plant was installed in 1893.

Table I—Summary of Filtration Plants in Pennsylvania as of June 1, 1928

Type plant	No. plants	No. units	Million gals. daily Rated capacity	Water filtered	Population in district	No. of Consumers
Rapid	164	685	487.50	270.67	2,139,731	1,856,910
Slow	10	38	41.60	34.30	223,829	220,365
Slow and rapid	5	106	119.50	90.80	627,898	585,000
Preliminary and slow	4	91	258.50	238.50	1,543,500	1,393,400
Pretreated and slow	1	56	200.00	121.00	690,000	640,000
Coke and rapid	1	1	0.20	0.20	62,327	1,800
Pressure	9	27	12.13	6.44	88,950	69,950
Total	194	1,004	1,119.43	761.91	5,376,135	4,767,425

The growth by years is shown in the following table:

Record of Filtration Plants Installed According to Year First Built to June 1, 1928

Year	Plants No.	Year	Plants No.	Year	Plants No.
1889	2	1903	5	1917	8
1890	..	1904	6	1918	4
1891	..	1905	7	1919	4
1892	1	1906	3	1920	7
1893	..	1907	2	1921	9
1894	1	1908	8	1922	2
1895	3	1909	5	1923	8
1896	3	1910	11	1924	6
1897	2	1911	7	1925	9
1898	2	1912	7	1926	5
1899	..	1913	7	1927	7
1900	2	1914	16	1928	6
1901	2	1915	6		
1902	6	1916	5	Total	194

The growth by periods is well shown in the table below. It will be noted that the greatest number of plants were built from 1910 to 1919, but in terms of rated capacity the previous decade totaled three times the amount with only slightly more than one-half as many plants. However, this period includes the Pittsburgh and all of the Philadelphia plants, except Queen Lane. It means that 1910-1919 witnessed the extension of filtered water to many of the smaller communities outside the largest centers of population. This, likewise, was true, although to a lesser degree, from 1920 to 1928.

Installation of Plants by Periods

Type of plant	Period—Years exclusive 1889 1899	1900 1909	1910 1919	1920 1928 (June 1st)
Rapid	11	33	60	58
Slow	1	4	8	....
Slow and rapid	....	2	....	....
Preliminary and slow	....	4	1	....
Pretreated and slow	....	1	....	....
Coke and rapid	....	....	1	....
Pressure	2	2	5	1
Total plants	14	46	75	59
Rated capacity million gallons daily	38.35	559.11	199.32	104.21

NOTE: In above table plants are listed according to first year in which plant is built—new plants only.

As a contrast to the slow growth at the beginning, only 14 plants in the first 10 years, 16 plants were built in the last two years having an aggregate rated capacity of 60 m.g.d.

A summary of filtration plants as of June 1, 1928, is shown in Table I.

On June 1, 1928, there were in Pennsylvania 194 filtration plants, almost all of which are in daily operation, serving four and three-quarter million people and producing in round figures 760,000,000 gal. of water every 24 hours. These plants combined have a rated capacity of slightly more than 1,100 mil. gal. daily. In addition to being filtered the water is, with very few exceptions, also chlorinated.

Cost of Ice Water Drinking Fountains.—The Water Board of Cambridge, Mass., maintains nine ice water drinking fountains during the summer months. The average cost per day per fountain of furnishing the ice was $1.75 in 1928; $1.63 in 1927 and $1.75 in 1926.

Accounting for the Water Pumped at Dubuque, Ia.

The percentage of salable water and pumpage accounted for by the Water Works of Dubuque were as follows for the year ending March 31, 1929:

	Gal.	Per Cent
Salable	620,689,500	63½
No revenue	357,398,500	36½
Accounted for	713,516,500	73
Unaccounted for	264,572,500	27

The following table from the 29th report of the Water Department, of which John W. McEvoy is Superintendent, shows the uses of the water pumped during the year:

		Gal.
Salable water		620,689,500
General flush of distribution system		6,500,000
Flushing 1,218 hydrants at 7,500 gal. each		9,135,000
Extension and plumbers' ditches		2,000,000
Service leaks, 96 to 100,000 gal. each		9,600,000
Eight hydrants broken (30 min. at 1,000 gal. per min.)		240,000
Cleaning level reservoir (3 times)		1,500,000
Draining West Third St. Reservoir		4,000,000
Known leakage in West Third St. storage reservoir (24,000 gal. per day)		8,760,000
Meters tested at 35 gal. each		200,000
Eagle Point Station		2,000,000
By Pass from Low Pressure to Level Reservoir		2,592,000
Flushing city streets and sewers	20,000,000	
Two horse fountains and 8 bubblers	8,800,000	
Fire department	8,500,000	
City parks (120 days at 10 hours)	3,000,000	
Bathing beach and athletic field	1,500,000	
Ice rinks (seven)	4,500,000	
Free water furnished the city		46,300,000
Total accounted for		713,516,500

Pumpage accounted for divided by the actual pumpage equals the per cent of pumpage accounted for; 713,516,500 divided by 978,088,000 equals 73 per cent.

Control of Filter Washing

Studies Show That 2.7 Ft. Vertical Rise Most Economical

By EDWARD S. HOPKINS
Principal Sanitary Chemist, Bureau of Water Supply, Baltimore, Md.

DUE to the serious clogging of sand filter beds in different parts of the country there is an opinion among many plant operators that the accepted standard wash of 2 ft. vertical rise per minute is not sufficient to keep the beds in proper condition.

This study was undertaken for the purpose of learning, if possible, some relation between washing characteristics and maintenance of clean beds with subsequent long periods of operating service. It being assumed that clean beds would minimize the possibility of mud ball clogging.

Clarity as Criterion of Cleanliness.—When washing filters it is the usual practice to utilize the clarity of the final wash water as the criterion of the cleanliness of the bed. This clarity being determined solely by visual inspection of the operator as the water passes into the drain troughs. Testing 7 experienced filter operators it was found that the turbidity of such wash water varied from 25 to 150 p.p.m., depending upon individual psychology and light conditions. Since it is logical to expect the period of filter operating service (filter runs) to be more or less controlled by the cleanliness of the bed, therefore the cleaner the bed, the longer the "filter runs" with other conditions being equal, due to a minimum initial mud load. This assumption is confirmed by a study of the curves in Fig. 2. Using a constant filtration rate (125,000,000 gal. per acre per day) with the applied water properly coagulated, having a turbidity between 4 and 7 p.p.m., it is shown that a minimum final wash water turbidity gives a maximum period of operation.

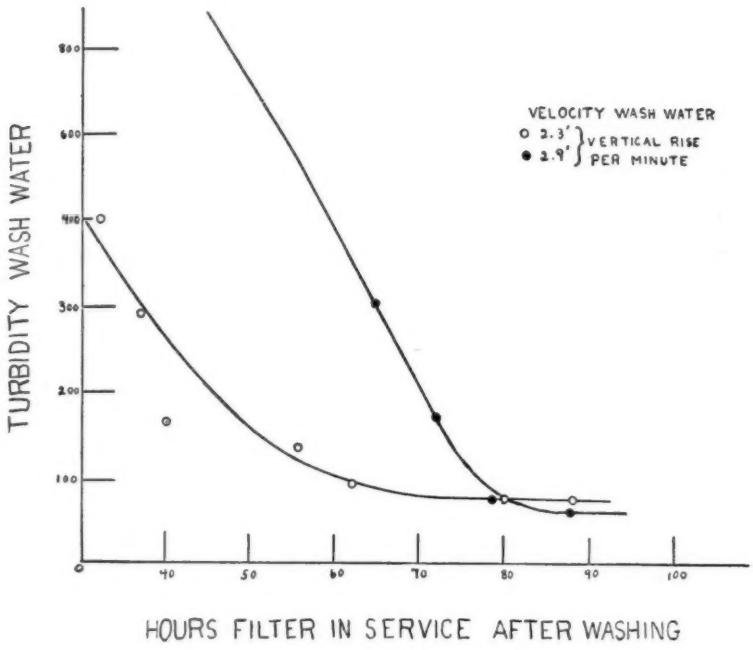


Fig. 2

Velocity a Factor.—These curves also indicate that the velocity of wash water is a factor in obtaining clean beds. When washing with a velocity of 2.9 ft. vertical rise per minute it will be noted that with a 1 minute wash and final wash water turbidity of 800 p.p.m. it was possible to obtain a filter run of 50 hours. Using a velocity of 2.3 ft. vertical rise, giving a turbidity of 400 p.p.m. for a similar period, a run of only 35 hours was obtained.

Further inspection of these curves show that the "filter runs" are practically constant for a final wash water turbidity of 75 p.p.m., thereby indicating that the bed is clean, with this condition existing for all velocities of wash water applied. Any increase in filter operating time greater than indicated, is due to the characteristics of the applied water rather than to washing conditions. These results confirm the belief that filters should be washed with the highest velocity comparable to plant design.

Simple Control Method.—Since it is not practicable as routine procedure to make turbidity determinations of the wash water during the restricted washing period a simplier control method must be utilized. Such a control can be easily maintained by regulating the time of wash in relation to the velocity of wash water applied as shown in Fig. 1. Inspection of these curves will show that the maximum cleanliness of bed (75 p.p.m. final wash water turbidity) does not require prolonged washing. Filters washed in accordance with these curves would be in this ratio:

2.3 ft. vertical raise per minute, requires a 6-minute wash,
2.7 ft. vertical raise per minute, requires a 4-minute wash,
2.9 ft. vertical raise per minute, requires a 4-minute wash.

A 4-minute wash at 2.7 ft. vertical rise is, therefore, the most economical rate, thereby giving an easily controlled cleanliness of bed, with assurance that the maximum "filter runs" are being obtained. These results would indicate that the present accepted standard of 2 ft. vertical rise is inefficient and if plant design will permit the use of these higher velocities, without loss of sand, increased "filter runs" will be obtained.

Acknowledgment.—The above is a paper presented at the 1929 Conference of the Maryland Water and Sewerage Association.

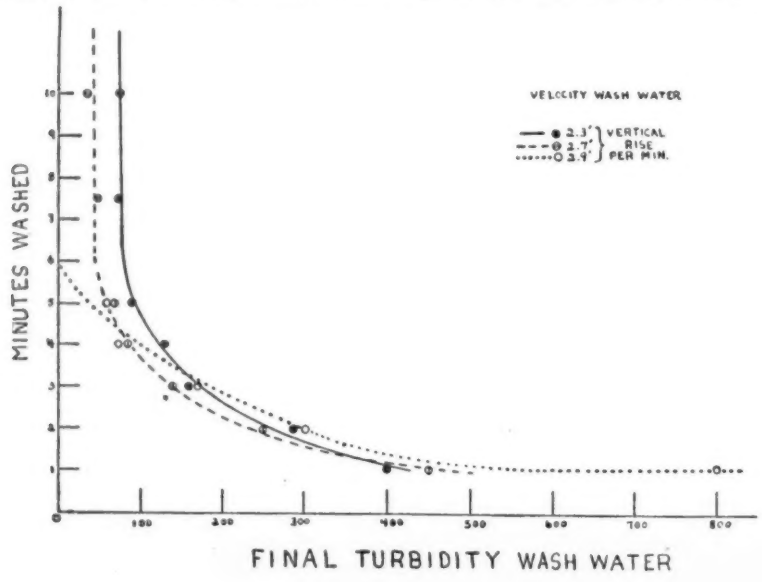


Fig. 1

Methods and Costs of Day Labor Construction of Water Mains and Sewers

Organization, Equipment and Methods Used by a Sanitary District in Maryland, and a Comparison of Cost of Machine and Hand Work in Trenching

By H. ROLAND DEVILBISS

Department Engineer, Washington Suburban Sanitary District, Hyattsville, Md.

THE Washington Suburban Sanitary District lies partly in Prince George's and partly in Montgomery County, Maryland, adjacent to the District of Columbia, and has an area of about 94 square miles. In addition, Gaithersburg and Washington Grove, lying in the central section of Montgomery County and embracing an area of about one square mile, form a detached part of the Sanitary District.

Organization.—The office of the chief engineer, who has charge of all engineering activities, is divided into two main departments, construction and maintenance and operation, and four divisions, design and drafting, surveys, plumbing, and accounts. Each is under a department or division engineer, responsible to the chief and deputy chief engineers.

The construction department engineer has direct charge of the construction of all water and sewer lines, filtration plants, water and sewage pumping stations, sewage disposal plants, water and sewer connections, purchase of construction equipment, small tools and supplies, and the inspection and handling of all construction material. Working directly under the department engineer are two resident engineers, one in charge of the eastern district and the other the western district, who supervise the work of the day labor forces on main lines and house services, and also of contract work. Assisting each of these men is a line and grade party, consisting of an instrumentman and rodman, which sets lines and grades and takes final measurements of completed work. On contract work inspectors are furnished these districts, whose duties are to see that the work is carried along in strict accordance with the specifications. An assistant engineer in the office has charge of house connection records, sketches, and the issuance of permits; and in addition looks after the office work of the plumbing division. At the warehouse is a storekeeper and an assistant whose duties are to inspect and check the receipt of all materials and their delivery to the various projects to be constructed. Working under these men are mechanics, machine operators, blacksmiths, carpenters, truck drivers and a street repair gang. Machine operators on assignment to a district work directly under the resident engineer. A final inspector and his gang report to and work directly from the department engineer's office.

The size of the day labor forces varies in accordance with the volume of work to be completed during the year. The maximum number of day labor gangs on water and sewer lines has been 11 and the minimum 2, while the maximum number of laborers employed on this class of work has been 396 and the minimum 21. On water and sewer house connections the maximum number of gangs has been 5 and the minimum 1, while the maximum number of laborers has been 79 and the minimum 13.

Work Accomplished.—During the past ten years this organization has constructed or supervised the construction of 190 miles of water lines, ranging in size from 1¼ in. to 24-in., 113 miles of sewer lines, ranging in size from 6-in. to 30-in., two water filtration plants and pumping stations, one having a capacity of 3,500,000 gal. per day, the other a capacity of 1,000,000 gal. per day, several large standpipes and elevated tanks, a sewage disposal plant having a capacity of 250,000 gal. per day, and four small sewage pumping stations, two each with a pumping capacity of 70 gal. per minute, another with a pumping capacity of 350 gal. per minute, and another with a pumping capacity of 300 gal. per minute, at an approximate total cost of $5,000,000. In addition 9,114 water services, ranging in size from ¾ in. to 8 in., and 6,368 sewer services were built.

The above figures do not include existing water lines, sewers, and water and sewer connections, purchased and incorporated into the district's systems at the beginning of operations, which amounted to approximately 53 miles of water mains, 56 miles of sewers, 1,586 water services, and 1,650 sewer house connections; nor do they include the expenses incurred in the purchase of these systems, or in the replacement of many old lines, the lowering of main services, and other incidental capital costs.

Equipment.—The hauling equipment consists of five 3½-ton trucks, three 1-ton trucks, three 1-ton dump trucks, and one 1-ton truck with dump body. In addition, four 1-ton trucks are equipped for use by the water and sewer house connection gangs.

Our excavating equipment consists of four trenching machines, two having a digging depth of 12 ft. and a maximum width of 33 in., one having a digging depth of 10 ft. and a maximum width of 30 in. and the other a digging depth of 7 ft. at 24-in. width and a digging depth of 5 ft. at 42-in. width.

Where one class of pipe line work is to be constructed and depths do not vary widely, the purchase of excavating equipment becomes a simple problem. In this district the problem is quite different. The territory is extensive and in most cases the lines authorized for construction by the commission involve both water mains and sewers in a single project. The average length of these installations is probably not greater than 800 ft.

For backfilling purposes we have four machines on full caterpillars, and equipped with boom attachments. In addition to refilling, these machines are used in lifting large boulders and rocks from trenches, unloading pipe from cars, lowering pipe into trenches, and withdrawing shoring from trenches.

We have three compressors, one having a capacity of 165 cu. ft. per minute, another a capacity of 90 cu. ft. per minute, both mounted on steel wheels, and another having a capacity of 90 cu. ft. per minute, mounted on a truck. The two mounted on steel wheels are moved from job to job on our trucks and are used where heavy rock cuts are encountered, while the one mounted on the truck is generally used in breaking concrete and macadam, and on trench work where a light rock cut is found. With the latter machine, speed of transportation is the big item and this compressor has proven valuable in breaking concrete and macadam for service connections. We are well equipped with concrete breakers, jackhammers, tampers and spades.

We have one 1½-ton crane mounted on a 7-ton truck and equipped with a ½ cu. yd. clamshell bucket. This machine is used in unloading cast-iron pipe and fittings from freight cars, loading trucks in our store-yard, unloading and handling crushed stone, gravel, cinders, coal, and in lowering large cast-iron pipe into trenches.

The crane was used successfully in excavating approximately 2,000 ft. of trench, averaging 8 ft. in depth, for a 30-in. reinforced concrete sewer. This work was in close proximity to a stream

and in marshy land. The machine was moved in while the ground was frozen, placed on a plank platform on the center line of trench, and carried on this platform throughout the work, which terminated at a paved roadway.

The tractor equipment consists of four small tractors. These are used mostly in moving construction materials and equipment to jobs which are not accessible to truck deliveries. Practically all of our trunk sewer work and some of the cross-country water lines fall into this class. In emergencies the maintenance and operating department has used the tractors in driving low-lift centrifugal and deep well pumps.

For trench pumps we have, mounted on steel wheels, eight 2-hp. gasoline-driven diaphragm pumps with 4-in. suctions, having a rated capacity of from 8,000 to 10,000 gal. per hour. We have also a number of 3-in. diaphragm hand pumps.

Type of Pipe for Water Mains.—The distribution system has been constructed of a number of different types of cast-iron pipe, among which are sand-cast, deLavaud centrifugal, sand spun centrifugal, and McWane horizontally cast, without precaulked joints. The sand-cast pipe, in sizes up to 24 in., and the deLavaud centrifugal, in sizes up to 20 in., have been in 12-ft. lengths, with the exception that during 1920 a small percentage of the sand-cast was in 5-meter lengths. The sand spun centrifugal pipe, in sizes up to 12 in., has been in 16-ft. and the McWane pipe, which we have used only in the 6-in. size, has been in 16.5-ft. lengths. Nearly all of this material is purchased under competitive bidding.

No provision is made in our contracts for a differential on 16-ft. or 16.5-ft. lengths over 12-ft. lengths. During the past year, at the direction of the chief engineer, I prepared an estimate of the average saving per linear foot in laying 16.5-ft. lengths instead of 12-ft. lengths, on sizes ranging from 6-in. to 12-in., and the results are shown in Table I.

We have recently tried some 12-in. deLavaud pipe in 18-ft. lengths. It is too early to form a definite opinion of its advantages over 12-ft. lengths, but it is not suitable for curved pipe lines and there is difficulty in handling where trenches have to be braced.

Joints.—For jointing cast-iron mains, braided hemp packing and leadite are used. On lines crossing bridges a sleeve is placed near the center of the crossing and this joint, as well as joints on lines passing under streams, is poured with lead. In the former case the lead joints make additional provision for expansion and contraction in the line, and in the latter it has been found that lead in these locations is easier to repair, if necessary, than leadite. Some of the bridges on which we have water mains are subject to considerable vibration from heavy traffic. The leadite joints on bridges, however, have shown little leakage.

Fire Hydrants Have Sliding Frost Casing.—The type of fire hydrant used was designed by Mr. Robert B. Morse, Chief Engineer of the district. It has an attractive appearance and the internal parts are adapted from the standard Mathews hydrant manufactured by R. D. Wood & Co. One of the unusual features embodied in the design of the hydrant is a sliding frost casing, which for our purposes has a maximum range of 2 ft., although if desired the casing could be manufactured to give either a greater or smaller range of adjustment. In this district, where water lines are constructed in a majority of instances in advance of permanent grading, the adjustable frost case has proven to be of untold value. Hydrants manufactured for the district with the sliding frost casing are for 4½, 5, 5½ and 6½-ft. depths of trench. If the ground surface at the time of placing the hydrant required a 4½-ft. hydrant, the casing would be in a fully collapsed position with its top just above the ground, while if at a later date a fill was made and a 6½-ft. hydrant was required the barrel of the 4½-ft. hydrant would be removed, a 6½-ft. barrel inserted, and the frost casing extended to its full extent so as to allow its top to protrude slightly above the finished grade. The operation would be reversed if a 6½-ft. hydrant was originally required and the street was graded off sufficiently to make a 4½-ft. hydrant suitable. Intermediate lengths of hydrant, of course, are used as required by conditions. Only a slight amount of digging is necessary in adjusting sliding frost casings to new grades, materially reducing the expense involved in changing other types of hydrants.

In placing fire hydrants a pocket 18 in. deep and 24 in. wide is excavated directly beneath the location of the hydrant, and this space is filled with stone, gravel or brick bats. This bedding affords a solid foundation for the hydrant and also allows proper drainage. After the hydrant has been placed and well braced back of the elbow, it is surrounded with the same class of previous material to a point well above the drip.

All hydrants have a main valve opening of 5¼ in. Hydrant leads are 6 in. in diameter and leads off from mains larger than 6 in. have a valve placed on them three feet from the face of the hydrant.

Laying Water Mains.—Water mains are laid with nearly the same precision, as to grade, as are our sewer lines. The construction department is furnished with a profile of the street, together with the elevations of the invert of the line at the various stations. Grade lines are set on the surface by the line and grade party and the foreman is furnished with a cut sheet which is followed in the laying of the main. Each pipe is graded with a rod in the same manner as sewers are graded, while the alignment of the main is generally done by eye, but with great care, nevertheless.

During the first few years the depth at which mains were to be laid was determined by the engineer in the field. Most of the streets and roadways in which mains were built were not at finished grade, and sufficient information was not available in the field to form a basis for reliable judgment as to the probable future condition. The result was that in some cases we have had to return and lower the mains, while in others we have mains deeper than necessary, due to heavy fills subsequently made. At a later date, the construction plans showed a reasonable finished street grade, as well as existing ground surface and the desired invert line for the main. The cuts from the existing ground surface to invert were scaled from the drawings in the field and the foremen attempted to dig to the required depths. Although this method was better than the crude one originally adopted, and generally used elsewhere, it was far from satisfactory and was in turn superseded by the plan first described.

Lowering Mains.—During the past ten years we have lowered 2,200 ft. of 4-in., 2,500 ft. of 6-in., 6,500 ft. of 8-in., 56 ft. of 10-in., 50 ft. of 12-in. and 550 ft. of 16-in. water main, most of which lengths, however, were in the systems purchased.

No difficulty has been experienced in lowering mains with either lead or leadite joints. A great many lead joints after lowering require recaulking, while with care very little repair has been necessary with leadite joints. Once in a great while a leadite joint will leak slightly, in which case the material is cut out at this point and the void is caulked with lead wool.

Sewer Construction.—Our specifications for vitrified sewer pipe require that all pipe shall be what is commercially known as "first quality" pipe, shall be in 3-ft. lengths, deep and wide socket, and double strength on sizes from 15 in. up. They further require that the contractor furnishing the material must submit to the chief engineer the name or names of the firm or firms which are to supply the pipe, together with the manufacture or manufacturers and location of the factory. The right is also reserved to require

Table I

Size, In.	Leadite and Jute per ft., 12-ft. lengths	Leadite and Jute per ft., 16.5-ft. lengths	Saving on Materials, 16.5-ft. lengths	Estimated Saving on Labor, 16.5-ft. lengths	Total Saving in using 16.5-ft. lengths
6	$0.032	$0.023	$0.009	$0.005	$0.014
8	0.0451	0.0329	0.0122	0.008	0.0202
10	0.0563	0.0409	0.0154	0.009	0.0244
12	0.0717	0.0522	0.0195	0.010	0.0295

the contractor to furnish an affidavit from the manufacturer that all pipe furnished is No. 1 pipe and first class in every respect.

Laying Pipe.—One of the most important things in the construction of a sewer system is the proper alignment and grading of each line constructed. In our district each length of pipe as it is laid is tested for line with a plumb bob and with a grade pole for exact elevation. After completion of an extension the lines are gone over by an inspector whose duties are to clean the lines of any sand or mud deposits that may have gathered during construction, to see if the alignment and grades between manholes are correct, and to search for leakage and other defects that may be visible in the manholes and pipe lines.

Final inspection is made of both contract and day labor work and in very few cases has it been necessary to reopen a trench to correct for alignment and grade. Occasionally a crushed pipe is found and repaired.

Y-Branches.—At first we placed Y-branches on our lines for vacant lots as well as for the existing houses. Vacant lots, however, are often not developed as originally planned and we were not a great while in learning that efforts towards making intelligent provision for Y-branches to serve vacant lots were mostly wasted. Further, the expense of placing Y-branches was considerable and much of it was lost for all time.

We now place Y-branches to provide only for existing houses and cut in on the lines for additional connections, in the manner to be described later.

Joints.—The materials which we have used in jointing sewers are cement, Hydrolene-B, Jointite and G-K.

Cement was first used and probably successfully in dry trenches and at points where no fear from root penetration need be entertained. In wet trenches, however, where underdrains are not used and where difficulties are always encountered in keeping water down, it is practically impossible for a foreman to obtain consistently good cement joints. Again, for the engineer to be assured of a proper cement joint, as generally specified, backfill certainly should not be permitted until the cement has had the opportunity of taking a fairly good set. It is generally believed, or at least stated, that backfilling over cement joints may be carried on, with care, directly after pipe laying. Care in doing this consists of refilling over the center of the pipes between joints, the backfill finally reaching a point from which it gradually rolls down and covers the joint. Refilling then proceeds in the ordinary manner. All joints may not be disturbed by this method but if some of the recently made joints are opened up after being covered by several feet of backfill you will find in a great many cases that the cement which was applied outside of the annular space has been wasted. I firmly believe that cement can be used successfully only in dry trenches, where backfilling need not proceed until the cement has become hard, and at points where there is no danger from root penetration.

Tests for Joints in Wet Trenches.—Inasmuch as a large proportion of our sewers are laid in wet trenches, and owing to some differences of opinion as to the pouring of joints which were moist and in some cases partly submerged, a few tests were conducted to determine the character of joints that could be secured under these conditions. In making these tests 6-in. deep and wide socket pipe was used, having a well-defined scoring on the spigot and the inside of the bell. The joints were packed with untarred jute and a depth of 1½ in. was allowed for the bituminous material. Water in which the tests were conducted was about the same turbidity encountered in the average wet sewer trench.

First: A number of sections of two lengths each were placed horizontally, the joints were packed and the inside of the bell and the outside of the spigot were moistened by running a wet brush around them. The joint was poured without any difficulty whatever. After pouring the joint one end of the pipe was sealed and the joint was submerged under 2 ft. of water for a period of 18 hours. Infiltration was negligible, not amounting to more than a tablespoonful in this period. The bells were broken and left the material easily, and the joint was cut and pulled from the spigot just as easily. Slight adherence existed between the two surfaces but the material was well keyed into the scoring and this undoubtedly was preventing infiltration. The joint itself showed no imperfection.

Second: A number of joints were poured by different methods with the invert of the pipe submerged 2 in. These methods were:

A. The joints were packed and the pouring gate was made directly on top of the pipe. The material was then poured and allowed to run down on both sides of the pipe. In some cases we had a joint that showed a solid face, while in most of our attempts the material failed to run on the bottom of the joint. In those that showed a solid face we found that at the bottom only a thin crust existed on the face of the joint, the material failing to fill the bell at this point. There was no adherence between surfaces.

B. The pouring gate in this case was placed slightly to one side of the top of pipe and the poured material forced the water around ahead of it. This method showed some improvement and some very good joints were obtained. All joints showed perfect on the face, but we found, on examination, a few with imperfections which would have allowed considerable infiltration.

C. In this case a small opening was left between the runner and the face of the bell at the bottom of the joint. The gate was built on top of the pipe and the bituminous material running down on each side of the pipe forced the water out at the bottom. This opening was stopped quickly with a piece of clay as soon as the compound appeared and the pouring of the joint was then completed. A number of joints were poured in this way and in no case did we fail to get a good result. The adherence in this case was about the same as obtained with the surfaces moist.

Water and Sewer House Connections.—Water and sewer house connections are built by the district as far as the property line. Table II gives the charges for the several sizes of connections which have obtained since the work started. It will be noted that these charges are gradually increasing which can be largely ascribed to the fact that increased width of streets in new subdivisions and a greater number of paved streets have caused the cost of doing the work to become materially greater.

The costs include the tapping of the water main or sewer, the necessary piping and fitting to carry the service to the property line, and on water services the meter housing and the placing of the meter.

Water.—For water services we first used galvanized genuine wrought-iron pipe. This material was superseded several years later by 1¼-in. cast-iron pipe, and during the past two years we have used copper tubing for services ranging in size from ¾ in. to 1½ in. On services larger than these cast-iron pipe is still used.

While both water and sewer connections are desired to a property the water service is laid in the same trench with the sewer connection. This is accomplished by tamping the backfill over the sewer pipe to the point at which the service is to be placed and then wedging a few pieces of timber across the trench to support the tubing. If the applicant desires water service only and the main is on the opposite side of the street from the property to be served shafts are opened on both sides of the road and the connecting channel is made by means of a 2-in. earth auger made up in sections 2 ft. long. This method has been used successfully under roadways 20 ft. wide. The copper

Table II

	Water, Size						Sewer, Size	
Year	¾ in.	1 in.	1¼ in.	1½ in.	2 in.	Over 2 in.	6 in.	Over 6 in.
1919 to 1924	$25.00	$30.00	$35.00	$40.00	$50.00	At cost but	$25.00	At cost but
1924 to 1925	30.00	35.00	42.00	50.00	60.00	in no case	30.00	in no case
1925 to 1928	35.00	42.00	50.00	60.00	74.00	less than	35.00	less than
1928 to 1929	42.50	50.00	60.00	72.00	90.00	cost of 2 in.	42.50	cost of 6 in.

tubing is easily passed through the opening made by the auger.

Water mains are tapped at an angle of 45 deg. and the end of the copper tubing is bent into the shape of a goose neck and connected to the corporation cock by means of a copper to brass fitting. Where it is necessary to joint two pieces of tubing a copper to copper coupling is used. Meters are placed outside in a housing, either between the curb and sidewalk or just back of the sidewalk. At the meter housing the tubing is bent to an angle of 90 deg. and carried vertically up through the housing and connected to the inlet side of a meter yoke by means of a copper to iron fitting. From the outlet side of the meter yoke galvanized genuine wrought-iron pipe is used to the property line.

Water services are laid at a minimum depth of 4 ft. The meter housing for ⅝-in. and ¾-in. meters consists of 18-in. terra cotta pipe 2 ft. long on which is placed a hollow concrete truncated cone 18 in. by 15 in. by 18 in. high. The frame and cover rest on this cone, making the total depth of the housing 46 in.

The meter yoke is equipped with a cut-off valve on the inlet side, which eliminates the need of curb cocks and boxes. Yokes are so placed that the inlet and outlet of the meter are 18 in. below the surface.

Water services are divided into two classes, single and double. Single water connections are mostly ¾ in. in diameter and provide service for one house. A double connection provides service for two houses and is usually 1 in. in diameter to the under side of the meter yoke where it is divided by a 1-in. by ¾-in. by ¾-in. tee, the branches of which are connected to the inlets of the two meter yokes which are placed in one housing. From the outlet sides of the yokes ¾-in. pipes are extended to the individual property lines. This type of connection, as a rule, is located near the dividing line between the properties served.

Sewer Connections.—As previously stated Y-branches are placed only to serve existing houses. As vacant lots are developed provisions for tapping the sewers must be made. For this purpose on sewers from 8 in. to 15 in. in diameter we have been using what is commercially known as a Washington Standard Thimble-Y. The Thimble-Y is an extra heavy soil pipe fitting and is made in 5-in. and 6-in. sizes. To place the Thimble-Y it is necessary to cut an opening in the side of the sewer slightly larger than the diameter of the connection to be made. The Y has a thin spigot 1 in. in depth which is placed in above-mentioned opening and is prevented from protruding into the barrel of the sewer by a flange which fits neatly against the barrel of the pipe. A concrete collar is then placed around the Y and extends one foot on each side of the tap made in the sewer. Occasionally, a length of pipe is broken in cutting the opening but the percentage of breaks, as compared with the number of taps made, is small.

Following suggestions of the chief engineer our office has recently designed a much lighter and less costly, but just as serviceable, sewer thimble, which is now being manufactured and will be used hereafter.

On 6-in. sewers the line is broken and a regular terra cotta Y-branch is inserted.

All ordinary sewer house connections are 6 in. in diameter and, when possible, are so laid as to give a cover of at least 7 ft. at the property line, except where houses are on a terrace.

Sewer connections are divided into two classes, single and double, the single connection being constructed in the ordinary manner, while the double is laid as one connection to a point between the curb and property line, where it divides by means of a Y, one branch going straight to one property and the other branch going at an angle of 45 deg. to the adjoining property.

Costs.—Perhaps a discussion of recent costs will prove more interesting than will one of the costs of several years ago when wages and prices of materials were constantly changing. During the past two years these items have been fairly stable and therefore, in Tables III and IV are shown the volume of work done and costs for the years 1927 and 1928, respectively, on both water mains and sewer lines constructed by the district's day labor forces. In addition, these tables further show a comparison of costs of machine work versus hand work.

The costs shown in these tables include every expense incident upon the work—labor, materials, supplies, small tools, equipment, transportation, repaving, maintenance of trenches for a period of three months, engineering (including surveying and designing) and administration. On water mains the costs include foundry inspection, fittings, valves, valve boxes, fire hydrant leads and fire hydrants; and on sewer lines they include manholes, manhole steps and manhole frames and covers—in other words, the lines complete. The costs per lineal foot include, therefore, every item of expense incurred from the time the project is authorized for construction by the commission until it is completed and placed in operation.

In 1927, 90 water projects were constructed having an average length of approximately 591 ft., and 58 sewer projects having an average length of 382 ft.; while, in 1928, 119 water projects were constructed having an average length of approximately 755 ft., and 66 sewer projects having an average length of 837 ft. From this one can readily see the number of times our gangs are moved during the course

Table III—Comparison of Cost Machine Work Versus Hand Work—1927

Size	Method	Water: Feet Laid	Water: Total Cost	Water: Cost per Ft.	Sewer: Feet Laid	Sewer: Total Cost	Sewer: Cost per Ft.
2 in. and less	Machine	934	$ 1,084.06	$1.160	.....	.....	.....
	Hand	674	960.75	1.427	.....	.....	.....
3 in.	Machine	102	187.59	1.838	.....	.....	.....
	Hand	538	1,009.21	1.875	.....	.....	.....
4 in.	Machine	.....	.....	.....	.....	.....	.....
	Hand	.....	.....	.....	.....	.....	.....
6 in.	Machine	17,614	33,688.19	1.912	4,683	$ 7,887.52	$1.684
	Hand	4,357	13,200.78	3.029	3,061	7,840.20	2.561
8 in.	Machine	9,025	24,582.54	2.723	8,235	15,124.98	1.836
	Hand	2,430	8,495.46	3.495	1,670	5,891.50	3.587
10 in.	Machine	5,881	18,938.97	3.220	1,813	3,682.60	2.031
	Hand	936	4,197.59	4.484	697	3,828.80	5.493
12 in.	Machine	1,643	6,370.61	3.877	.....	.....	.....
	Hand	4,310	18,075.13	4.293	.....	.....	.....
16 in.	Machine	352	1,555.39	4.418	.....	.....	.....
	Hand	3,900	19,120.35	4.903	.....	.....	.....
20 in.	Machine	648	5,476.36	8.451	.....	.....	.....
	Hand	.....	.....	.....	.....	.....	.....
24 in.	Machine	.....	.....	.....	.....	.....	.....
	Hand	.....	.....	.....	2,011	20,721.32	10.304
Totals and averages	Machine	36,199	$91,883.71	$2.538	14,731	$26,695.10	$1.805
	Hand	17,044	65,059.27	3.818	7,439	38,281.82	5.146

Table IV—Comparison of Cost Machine Work Versus Hand Work—1928

Size	Method	Water: Feet Laid	Water: Total Cost	Water: Cost per Ft.	Sewer: Feet Laid	Sewer: Total Cost	Sewer: Cost per Ft.
2 in. and less	Machine	992	$ 1,413.02	$1.526	.....	.....	.....
	Hand	516	644.16	1.248	.....	.....	.....
3 in.	Machine	87	111.79	1.282	.....	.....	.....
	Hand	388	1,041.52	2.684	.....	.....	.....
4 in.	Machine	2,024	2,520.47	1.245	.....	.....	.....
	Hand	544	1,163.83	2.139	.....	.....	.....
6 in.	Machine	37,582	64,675.55	1.720	19,138	$31,700.61	$1.656
	Hand	1,010	3,288.64	3.256	907	2,583.88	2.848
8 in.	Machine	30,361	64,596.93	2.126	27,656	53,039.99	1.918
	Hand	1,075	2,376.73	2.289	681	1.570.13	2.305
10 in.	Machine	9,357	24,677.96	2.637	5,331	10,191.92	1.912
	Hand	123	422.44	3.434	.....	.....	.....
12 in.	Machine	3,605	14,219.71	3.944	324	1,064.01	3.284
	Hand	1,369	6,612.40	4.830	.....	.....	.....
15 in.	Machine	.....	.....	.....	.....	.....	.....
	Hand	.....	.....	.....	1,259	5,835.84	4.635
16 in.	Machine	863	3,541.34	4.103	.....	.....	.....
	Hand	.....	.....	.....	.....	.....	.....
Totals and averages	Machine	84,871	$175,756.77	$2.083	52,449	$95,996.53	$1.830
	Hand	5,025	15,549.71	3.094	2,847	9,989.85	3.509

of a year, and moving from one project to another involves considerable expense, not only in the actual cost of moving, but also in the loss of time. Where possible, the move is made after working hours and the latter loss eliminated, but in the majority of cases the move, unfortunately, has to be made during hours.

On water mains our standards require a minimum cover of 4 ft. In a great many sections of the district the streets have not been cut down to the final grade and the mains consequently have to be laid at an excessive depth, anticipating that some time in the future the street will be brought to a finished grade. While we do not classify our excavation depths on day labor work, the writer feels safe in saying that the average depth of water trenches on all sizes of water mains is at least 6 ft. Naturally, these ungraded streets increase the depths of the sewers and frequently to an amount greater than our machines will excavate, thereby making it necessary to handle this deeper excavation by hand.

In the northern and western sections of the district considerable rock is encountered, while in the southern and eastern sections we have to contend with caving ground, water, silt and quicksand.

Machine Work Versus Hand Work.—Some work on machine jobs necessarily has to be done by hand; for example, the crossing of gas mains and services, culverts, tunneling of improved roadways and operations in the proximity of trees, poles, etc. This kind of hand work is included in the tabulated machine costs, while in the hand work costs no machinery whatever was employed.

In Table IV it will be noted that machine work on 2-in. mains and smaller costs $0.28 more per foot than did hand work. Only parts of these jobs could be done by machine on account of rock. This, together with the small volume of work done, caused the cost to be exceedingly high. Again, in Table III, it will be noted that the cost of a short line of 20-in. water main was $8.45 per foot. This job was located in the eastern section of the district, in an ungraded street, which required the main to be laid at an average depth of 8 ft. At a depth of 3 ft. quicksand and water were encountered, necessitating the balance of the excavation to be done by hand, tight shoring driven, and pumps operated 24 hours per day. In addition to these troubles, a rainy period of several days occurred during the course of construction. These cases are mentioned merely to show conditions frequently met with in the district. Similar cases could be cited on other sizes of lines.

Acknowledgment.—The foregoing is an abstract of a paper presented at the 1929 Conference of the Maryland Water and Sewerage Association.

A New Lining for Cast Iron Pipe

The engineers of the Wailes Dove-Hermiston Corporation, working in cooperation with the engineers of the United States Pipe & Foundry Company, have recently developed an efficient method for lining cast iron pipe with their bitumastic material. Although the bitumastic lining is not a new development in the field of pipe construction, except in cases of extreme necessity the difficulty of application of this lining to the inner surface of large diameter pipe has hitherto been a handicap to its widespread use. The old method of applying it with a brush was tedious, expensive and slow. The new method recently developed is efficient, rapid and economical.

The applications of this lining to the trade are numerous indeed. Bitumastic Lining is stated to be completely resistant to both concentrated and dilute acids, as well as to alkalis and corrosive liquids of all types and degrees. For piping systems which carry waters strongly impregnated with sulphur, and whose reaction is therefore slightly acid, the use of bitumastic lining is claimed to be ideal. When the water is alkaline, the corrosive effect on the bitumastic lining is stated to be nil.

In addition to the resistance of this lining to the various liquids which may be passed through the pipe, the smooth glossy surface is stated to present no lodging place for tubercles, barnacles or other growths with which it may come in contact. The smooth glossy surface, which resembles baker enamel, is claimed to permit the liquids to flow through the pipe with considerably less frictional loss than is the case with uncoated pipe, thus producing a greater pipe line efficiency.

The new method of applying this lining makes use of a new application of an old theory—that of centrifugal force. The bitumastic enamel is heated in a cauldron to the proper melting temperature. The pipe to be coated has previously been dipped in a priming coat and has been allowed to completely dry. The pipe is placed on rollers, driven by an electric motor, and is caused to rotate rapidly. The correct amount of bitumastic enamel, being now quite fluid, is poured into a steam-heated trough which is run into the rotating pipe and tipped over evenly, distributing the bitumastic lining along the entire length of the pipe. In a moment the material is distributed by centrifugal force over the entire inner surface of the pipe, and rotation is continued until the material has completely hardened. If a double lining is required, the second layer is put in before the first is entirely solidified. Thus, a layer of Bitumastic Enamel may be applied of any desired thickness without difficulty.

On Sept. 24th, a number of hydraulic engineers, who are especially interested in the elimination of corrosion or tuberculation on the inside of cast iron pipe lines, were invited to the Burlington Plant of the United States Pipe & Foundry Company to witness a demonstration of the application of the lining by this process. Fifty prominent engineers from New York, Philadelphia, Baltimore and points more remote responded. Two pipe were lined, both 48 in. in diameter, and both pipe were handled with ease and dispatch by the perfectly trained crew. The first pipe contained a standard lining approximately $\frac{3}{32}$ in. thick, while the second was lined with double thickness. Great admiration was expressed by all present over the efficient manner in which the work was done as well as over the excellence of the effect produced.

Due to the character of the water carried in certain sections of the country, tuberculation in pipe lines has, over a period of years, reduced the carrying capacity of these lines. Engineers are of the opinion that the use of the bitumastic lining will eliminate this trouble. An example of the efficiency of the bitumastic lining will be found in the parallel 36-in. and 42-in. lines which form the Narrows Siphons carrying the water supply to Staten Island, New York. The older of these lines has been in service over 15 years, carrying soft water. The last examination showed the bitumastic lining to be in excellent condition, and the lines have retained their full carrying capacity.

Silting of Reservoirs

In a discussion in Vol. 93, 1929 Transactions of the American Society of Civil Engineers, R. L. Meeker, Consulting Engineer, Denver, Colo., gives the following table summarizing the losses in various reservoirs in the southwest from silt deposition:

Silt Deposition, Channel Reservoirs, Southwestern United States

Reservoir	Stream	State	Drainage area in square miles	Original capacity, in acre-feet	Silt Deposition: Lost capacity, in acre-feet	Silt Deposition: Period, in years	Percentage of original capacity
Elephant Butte	Rio Grande	New Mexico	30,000	2,368,000	220,000	11-1915-1925	9
Roosevelt	Salt	Arizona	5,760	1,367,000	101,000	20-1906-1925	7
Austin	Colorado	Texas	38,000	*49,300	23,559	8-1893-1900	48
				32,029	30,552	13-1913-1926	95
McMillan	Pecos	New Mexico.	.....	28,732	12,232	10-1895-1904	42
Zuni	Zuni	New Mexico.	500	10,230	9,689	22-1905-1927	95
Castlewood	Cherry Creek	Colorado	165	5,267	†	38-1890-1927	†
Buckhorn	Buckhorn Creek.	Colorado	130	1,191	565	18-1907-1925	47

*Old reservoir.
†Precise data are not available at this time.

New Water Supply for Los Angeles

Surveys Now Under Way for Supply from Mono Basin

SURVEYS are being made by engineers of the water department of Los Angeles, Calif., to determine the feasibility of securing an additional water supply from the Mono Basin to tide the city over until a supply from the Colorado River is available. The plan, in brief, according to the Los Angeles Times, from which these notes and the illustration are taken, provides for collecting the water in a reservoir on Rush Creek, its transportation through a tunnel into the Owens Valley water shed and thence into the Los Angeles aqueduct system, via the Owens River. An alternative plan routes the collected water around the south end of Mono Lake, through Adobe Valley and down into the Owens Valley near Bishop, via Fish Slough. It is expected the engineers' report on the project will be completed by the first of the year.

The Mono Basin is just north of Owens Valley, being separated from the valley by a formation of volcanic hills. The water available in the basin is estimated at from 180 to 235 second feet. The streams flowing into this basin from the High Sierras include Parker Creek, Walker Creek, Leevining Creek, Rush Creek and possibly Mill Creek. All plans call, first, for the collection of the flow of the above streams into a common channel. This channel, which will start from Leevining Creek, would consist of a series of open canals and tunnels intercepting the streams. It would deliver the water into a proposed reservoir on Rush Creek, formed by the erection of a 150-ft. concrete dam.

Later the collecting channel that picks up the water from the streams may be extended north beyond Leevining Creek to tap Mill Creek if economically feasible.

An 11-mile tunnel would tap the reservoir on Rush Creek and carry the water under the divide that separates Mono Basin from the Owens Valley, emptying it into Deadman's Creek, which forms the headwaters of the Owens River. The Owens River furnishes the bulk of the flow in the aqueduct, and so once the Mono water gets into the Owens River it is a part of the aqueduct system.

The Owens River flows through Long Valley and into the spectacular Owens River gorge, a very deep chasm 23 miles long. At the head of the gorge it is proposed to erect the Long Valley dam to provide storage capacity and also control the flow of the river through the gorge. The city has an easement on all the lands in the valley that would be covered by water with the building of a 100-ft. dam. Coming through the gorge there is a possible power development of about 86,000 hp. at three possible sites.

The cost of the development is estimated roughly at $30,000,000, this figure including the following:

Water rights: Southern Sierras Power Co., $8,000,000; private holdings, $2,000,000; $6,000,000 to get the water into the Owens River water shed, including cost of tunnels, dam and the like; and another $10,000,000 to buy up the remaining ranch lands and towns in the Owens Valley. In addition, there will be cost of enlarging the aqueduct to handle the combined Owens Valley and Mono basin waters.

MILL CREEK
TUNNEL AT THIS POINT MARKS BEGINNING OF MONO BASIN PROJECT
MONO INN
TIOGA PASS
SERIES OF SHORT TUNNELS AND OPEN CANALS PICK UP WATER FROM STREAMS AND DELIVER IT INTO RESERVOIR PLANNED ON RUSH CREEK
LEEVINING CREEK
MONO LAKE
WALKER CREEK
PARKER CREEK
GRANT L.
SILVER L.
RUSH CR.
150 FT. DAM CONTEMPLATED AT THIS POINT ON RUSH CREEK — TO BE TAPPED BY 11 MILE TUNNEL UNDER THE HILLS WHICH FORM THE DIVIDE BETWEEN MONO BASIN AND HEADWATERS OF OWENS RIVER.
ANOTHER PLAN CARRIES WATER IN OPEN AQUEDUCT AROUND SOUTH END OF MONO LAKE THROUGH ADOBE VALLEY, THENCE INTO OWENS RIVER NEAR BISHOP
11 MILE TUNNEL
GULL L.
JUNE L.
DIVIDE
ADOBE VALLEY
TUNNEL ENDS HERE — WATER EMPTIES INTO DEADMANS CREEK THENCE INTO OWENS RIVER TO BE PICKED UP BY PRESENT AQUEDUCT
DEADMANS CREEK
LONG VALLEY
LONG VALLEY DAM AND STORAGE RESERVOIR SITE
SHERWIN HILL
FLOWING THROUGH OWENS RIVER GORGE, WATER MAY BE USED TO DEVELOP 86,000 H.P. AT 3 POWER PLANTS
ROUND VALLEY
OWENS RIVER
FISH SLOUGH
LAWS
BISHOP
TO AQUEDUCT INTAKE NORTH OF INDEPENDENCE

Artist's Visualization of the Mono Basin Project

Sewage Discharge from Submerged Outlets

Salinity of Body of Water Receiving Sewage an Important Factor in Its Diffusion, Oxidation and Decomposition

By KENNETH ALLEN

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THE phenomenon of the course and diffusion of sewage discharged from submerged outlets is one that has rarely received close study, although the general results have been long recognized.

In inland cities the phenomena accompanying the discharge of sewage from submerged outlets in lakes and rivers will differ radically from those to be observed in salt water.

The effect of salinity was brought out very clearly in experiments made by the writer for the Metropolitan Sewerage Commission of New York in 1909. Owing to the configuration of the harbor, the flow of upland water brought down by the Hudson River, and the action of the tides, the percentage of sea water was found to average 74.8 at West Bank Light in the Lower Bay and 33.7 in the Hudson River off Fort Washington Point. The extreme readings noted at those points during the year were 93.2 per cent at West Bank Light on Nov. 5, and 0 per cent at Fort Washington Point on April 8. Conditions varied, therefore, from those of fresh water to those of nearly pure sea water.

Several series of experiments were made to learn more of the action of sewage discharged into waters of different specific gravities.

Experiments on Flotation.—Croquet balls were weighed until they balanced in water of a specific gravity of 1.000, and then released in water of known specific gravity at a given depth.* The time required for the balls to rise to the surface was noted with a stopwatch. (See Table I.)

The results, while not entirely consistent, give a fair idea of upward velocities of smooth solids released in waters of different density that will not be reached by a diffusing liquid. Bottles, released in a similar way, rose even more rapidly, on account of their smoother surfaces.

Laboratory Experiments in Diffusion.—A total of 31 experiments was made to observe the action of a dyed sewage or water when discharged from a pipette, or small tube, in waters of different salinity. Alcoholic solutions of eosin and fluorescein and aqueous solutions of bluing, permanganate of potash, and uranine were tried. The dye, especially the bluing, was found to increase the specific gravity slightly, and it was feared that alcohol might affect diffusion. Permanganate of potash was likely to be decolorized by organic matter. It was finally decided to use uranine ($C_{20}H_{10}O_5Na_2$) which, in a strong solution, is a deep red, but becomes a brilliant green on dilution, readily detected in a Nessler tube in a 1:20,000,000 dilution. A dye called "Special Scarlet" was used later and was found to be a very serviceable and less expensive substitute.

These experiments indicated clearly the effect of specific gravity and the way in which a drop or a jet diffused. Some of the forms observed were beautiful as well as interesting: Waves of floc; rings like smoke rings; funnels; and cones and wavy threadlike lines, rising or falling, and diffusing rapidly or slowly. The diffusion of 2 cu.cm. of a 1 per cent solution of uranine added gently to the surface of 200 cu. cm. of waters of varying salinity in beakers and jars began in a few minutes, was complete in an hour with tap water, and took 48 hours in a mixture containing 25 per cent of sea water, while it had not occurred with a 75 per cent mixture of sea water in a week's time. (See Table II.) Salinity, therefore, was shown to have a marked effect on diffusion.

Experiments in Diffusion in the New York City Aquarium.—These experiments, twelve in number, were made to confirm the results obtained in the laboratory before conducting others on a large scale in the open water. The tank was 4 ft. high, 4 ft. wide at the top, narrowing to about 3 ft. at the bottom, and 8 ft. long, with an overflow a few inches below the top. The dye and water were fed from a funnel and by a tube to a nozzle, the depth and direction of which were adjustable. Nozzles of ¼ in., ⅜ in., and ½ in. were used.

Without going into the details of these experiments, some of the results were as follows:

(a) With a horizontal jet a considerable current may be induced at a slight depth without apparent effect on the surface itself.

(b) With a ¼-in. horizontal jet of fresh water in fresh water, under a head of 1.0 ft., the axis of the jet remained horizontal, enlarging in diameter to 3 in. at a distance of 1.25 ft. from the nozzle and to 6 in. at a distance of 3.0 ft. from the nozzle, and then diffusing slowly. After substituting a jet of sewage, the diameter 3.0 ft. from the nozzle was 12 in.

(c) In the case of a ¼-in. upward jet of fresh water at a depth of 3.0 ft. in fresh water under a head of 2.5 ft., the jet formed a concave cone 5 in. in diameter 1.0 ft. from the nozzle and spread in a thin layer at the surface. By the end of 5 min. diffusion was quite uniform to a depth of 1.5 ft.

(d) With a ½-in. horizontal jet of sewage in fresh water near the bottom of the tank, 3.4 ft. below the surface and under a head of 0.15 ft., there was a gradual descent of the jet to the tank bottom which it covered with a blanket about 2 in. thick, with very little diffusion, in 3 min. The temperature of the sewage was 62° Fahr., with a specific gravity of 1.001. The temperature of the water was 61° Fahr., and its specific gravity was 1.000.

(e) With a ½-in. jet (nearly horizontal) of sewage in harbor water, at a depth of 3.3 ft. and under a head of 0.15 ft., a temperature of sewage 62° Fahr., specific gravity, 1.0015, and a temperature of water, 58° Fahr., the effluent rose promptly in an upward curving cone, reaching the surface in 1 min., where it spread out. At the end of 3 min. the supply of dye was cut off and by the end of 5 min. there remained but a thin green layer at the surface.

(f) For a ¼-in. horizontal jet of sewage in harbor water, as in Result (e), under a head of 1.0 ft., the diameter in a plane 1.0 ft. above the nozzle was 8 in. The horizontal component of the jet did not appear to affect the time taken to reach the surface.

Table I—Experiments to Determine Upward Velocity of Smooth Solids

Experiment No.	Number of tests	Specific gravity of water	Vertical distance traveled, in feet	Average velocity, in feet per second
1	13	1.0215	1.47-1.51	0.500
2	10	1.016	1.70	0.455
3	15	1.007	1.62	0.252
4	15	1.009	1.62	0.301
5	4	1.0195	10.0	0.655
6	10	1.0215	10.0	0.49

*The salinity of sea water varies in different parts of the ocean, but for this region chlorine of 18,000 parts per million, corresponding to a specific gravity of 1.025, was assumed as constituting 100 per cent.

(g) With a ¼-in. upward jet of sewage in harbor water, at a depth of 2.5 ft., and under a head of 1.0 ft., the sewage rose promptly to the surface with little diffusion and then spread in a thin layer. The diameter 2 ft. above the nozzle was 6 in.

(h) At a ¼-in. downward jet of sewage in harbor water, under a head of 1.0 ft., the jet rose as a column of greater diameter and with more rapid diffusion than in Result (g) and spread at the surface. Diffusion was increased materially by a slight agitation of the water.

The results of these experiments are generally confirmed by the work of the authors and by a similar apparatus, but their analysis has been carried to a point where the path of a jet of fresh water in sea water may be more closely predicted.

Experiments on Diffusion in New York Harbor.—Nine experiments were made by placing a dye in the surface water of the harbor, or at sewer outlets, to observe the rate at which sewage might be expected to become diffused and dispersed under different conditions. These were followed by 21 experiments to observe the result of discharging dyed water at various depths and locations in the harbor.

In three experiments at the outlet proposed at that time for the Bronx Valley Sewer, in the Hudson River, off Mount St. Vincent, dyed water was pumped at rates of 17 to 25 gal. per min., to depths varying from 5 to 25 ft. through a 1¼-in. hose. The velocity of the river water varied from 0 to 3 ft. per sec., and its specific gravity varied from 1.010 to 1.013. The detection of dye at the surface was uncertain at slack water from depths greater than 16 ft., or in a current of 2½ ft. per sec. from depths greater than 9 ft.

Nine similar experiments were made at various points in New York Bay, Gravesend Bay, and the Upper East River. These indicated that:

(a) With a difference in specific gravity of 0.004, dyed water will rise in quiet harbor water from depths of 10 to 26 ft. at a mean rate of 0.10 to 0.14 ft. per sec., and be visible at the surface.

(b) With a difference in specific gravity of 0.004½, it will rise from a depth of 18 ft. at a mean rate of 0.17 ft. per sec. and be visible at the surface.

(c) With a difference in specific gravity of 0.016, it will rise from a depth of 27 ft. in a current of 1½ ft. per sec. at a mean rate of about 0.10 ft. per sec., and be visible at the surface.

(d) In moderate currents dyed water may be expected to be visible at the surface if discharged at depths of 15 to 20 ft., but that the rate of ascent is variable. The limit from which it can be discovered under the most favorable conditions appears to be about 40 ft. In sea water (specific gravity, 1.021½), with a current of 0.3 ft. per sec., diffusion is very gradual at first, but proceeds more rapidly later.

Two similar experiments, but on a larger scale, were made in the Upper Bay, near the site of the Passaic Valley Sewer outlet off Robbins Reef, which has since been built.

In the first experiment, 16,500 gal. of a 1:2,750 solution of uranine in tap water was discharged 40 ft. below the surface through a 2½-in. hose, at a rate of 205 gal. per min., giving a velocity of 14 ft. per sec. The specific gravity of the harbor water varied from 1.0115 to 1.0135.

In the second experiment, 56,000 gal. of tap-water containing 290 lb. of uranine, forming a solution with a specific gravity of 1.003, was pumped from a pontoon through an 8-in. pipe, at a rate of about 2,320 gal. per min., or with a velocity of 14.8 ft. per sec., to a depth of approximately 40 ft., the pontoon and discharge pipe rising as the experiment progressed. The specific gravity of the harbor water was 1.015, and the velocity of its current, as observed by submerged floats, averaged: 0.45 ft. per sec. at a depth of 3 ft.; 1.03 ft. per sec. at a depth of 20 ft.; and 2.85 ft. per sec. at a depth of 50 ft.; from which the mean velocity at the point of discharge was taken as 2.2 ft. per sec.

In spite of these large volumes no evidence of the discharge was observed in either case, although it had been detected once before in one of the smaller experiments at this point and at the same depth. The inference was that diffusion occurred rapidly due to subsurface currents at the time of the larger experiment, and that the rise of sewage to the surface probably depends more on the velocity of such currents than on the volume of sewage discharged.

Table III—Effect of Depth on Percentages of Sea Water and Dissolved Oxygen Near Sewer Outlets in Boston Harbor, Massachusetts

Date, 1911	Distance from outlet, in feet	Outlet	Depth below the surface, in feet	Percentage of sea water	Dissolved oxygen percentage of saturation
August 29	10	Deer Island	1	68	53
			7	89	88
August 30	90	Deer Island	New surface to 1	80	80
			2	88	95
			3-60	94	100
September 6	1,000	Moon Island	New surface to 1	82	73
			1	82	75
			2	86	90
			3	88	95
			5	90	98
			10	92	99
			15	94	100
September 7	500	Moon Island	New surface to 1	80	68
			2	86	78
			3	88	88
			5	88	94
			10	90	97
			20	92	100
September 11	1,000	Moon Island	New surface to 1	78	68
			2	82	76
			3	86	87
			5	88	92
			10	92	97
			15	94	100
September 11	4,600	Moon Island	New surface to 1	80	72
			2	84	85
			3	88	92
			5	90	97
			10-20	94	100
September 14	2,000	Moon Island	New surface to 1	82	68
			2	86	73
			3	88	85
			5	90	92
			10	94	97
			15	96	100
September 14	2,500	Moon Island	New surface to 1	84	72
			2	86	80
			3	88	86
			5	90	97
			10	94	100
			15	96	100
September 18	5,280	Moon Island	New surface to 1	86	75
			2	88	82
			3	90	89
			5	90	97
			10	94	100
September 18	1,000	Moon Island	New surface to 1	82	68
			2	86	74
			3	88	85
			5	90	92
			10	92	97
			15	92	97

Table II—Diffusion in Waters of Different Salinity*

Liquid added	Top water	Sewage	Sewage
Dilution of uranine solution	1:100	1:1,000	1:10,000
Specific gravity of liquid added	1.000	1.001½	1.001½

Percentage of sea water	Percentage of tap water	Diffusion apparently took place between		
100	0	2 and 24 hours	6 and 48 hours	.....
75	25	2 and 24 hours	6 and 48 hours	Not complete at end of week
50	50	2 and 24 hours	6 and 48 hours	56 hours
25	75	5 min. to 2 hours	6 and 24 hours	48 hours
0	100	At once	At once	Few minutes and 1 hour

*Experiments by Dr. Payne B. Parsons, Bacteriologist, Metropolitan Sewerage Commission of New York.

It is clear that salinity of a body of water receiving sewage is an important factor in its diffusion, as it also is in its oxidation, its effect on precipitation, its decomposition, its general appearance, and its accompanying odors. The sewage of the city of Washington, D. C., discharged in 30 ft. of fresh water in the Potomac River and that of Hamburg, Germany, discharged at a depth of 30 ft. in the Elbe, is difficult to discern at the surface, while that from the Boston and New York outlets forms a field on the surface that is often clearly visible, well defined, and extensive. It was noted in the New York experiments with dyes that the edge of the field on the windward side was most clearly defined, while the more rapid diffusion took place on the leeward side.

From what has been said it is evident that diffusion is promoted by locating inlets in deep water of low salinity, but having a high velocity of flow. Discharge should be horizontal and from multiple outlets. The submerged outlets from Deer Island in Boston Harbor, with fourteen horizontal orifices, and the one hundred and fifty outlets of the Passaic Valley Sewer, in which the upward-flowing stream is dissipated by spiral flanges, are excellent examples of effective diffusion and dilution of an effluent in salt water.

As the visibility of sewage fields in calm weather is largely due to the oily surface film known as "sleek," it is obvious that for the sake of appearance, oily wastes should be kept out of effluents so far as practicable: First, by control and treatment at the source of industrial wastes of this nature; second, by providing municipal collection of crankcase wastes from garages; and, third, by installing skimming tanks on the line of outfall sewers. Other points that might be emphasized are the very great effect of turbulence of current on diffusion, and the variable effect of winds on the shape and extent of the sewage field.

Table III shows how the sewage discharged at the Deer Island and Moon Island outlets in Boston Harbor concentrates near the surface, as indicated by the increasing salinity or percentage of sea water and saturation with dissolved oxygen with increasing depth.

Acknowledgment.—The foregoing is a discussion in the September, 1929 Proceedings of the American Society of Civil Engineers.

Pipe Line Bridges; Methods of Covering Pipe and Costs

In the last two years the progress made in the installation of water mains in the Baltimore County Metropolitan District of Towson, Md., necessitated the crossing of three railroad right-of-ways, which were spanned by highway bridges, and due to the fact that the railroad company's and highway's engineers refused to grant permits for the installation of these water mains on the floors of the existing bridges, separate trusses were constructed to carry the mains across. In a discussion at the 1929 conference of the Maryland Water and Sewerage Association A. F. D. Domenics gave the following information regarding this work.

Table I

	79-Ft. Span	126-Ft. Span*	96-Ft. Span
Size pipe, inches	10	10	12
Type of covering	Box and Saw-dust	Box and Saw-dust	Felt and Galv. Iron
Cost of truss and extra steel	$ 846.56	$1,518.69	$1,425.00
Piers	.....	.....	487.50
Cost of laying pipe	30.02	62.40	195.84
Cost of covering	178.86	171.60	499.86
Total cost of job, exclusive of pipe and fittings	1,055.44	1,752.69	2,608.20
Cost per foot, dollars	13.36	13.91	27.16

*The actual length of this truss is 66 ft., although the total length of the crossing is 126 ft. Three brackets were placed 5 ft. apart on the side of the existing highway girder, so as to carry the pipe projecting from each end of the truss.

The Trusses.—The trusses that were accordingly designed and built were of the Warren Type, and were 79, 66 and 96 ft. in length. The first two mentioned were built and hauled to the site and placed on the existing abutments in one piece, but the latter, due to its length was hauled to the site in two sections, and was riveted together in the field and placed in position in one piece. The water mains on these trusses rest on 2x12x¾-in. plates placed equally along the span and are bolted on these plates by ¾-in. U-bolts. To safeguard against any overturning effect, the 66 and the 79-ft. trusses were attached to the existing highway bridges by five straps, three on top chord and two on bottom. These straps were riveted onto the truss, but on the existing bridges the holes on the straps are slotted and bolted to the structure. The reason for slotting is to transmit as little vibration to the truss as possible and at the same time hold the truss in place. The pipe fittings at both ends of the trusses are held secure from blowing out by pipe clamps. Lead was used for jointing material on the three jobs, and there also was placed a sleeve near the center of the crossing to provide for expansion and contraction in the main.

Protecting Pipe from Freezing.—Two methods of protecting pipe from freezing have been used by the department. The first is by encasing the pipe in a wooden box with saw-dust. The box coverings are built as follows: Frames made of 2x4-in. studdings are placed 3 ft. apart around the pipe. On the sides of these frames are nailed 1-in. boards and on the bottom 2-in. boards; this forms the box. The box is then filled with saw-dust, so that, the pipe has at least 4 in. of the material on sides and bottom. The cover is made of 1-in. lumber and then the entire box is covered with asphalt roofing paper, in order to keep the rain from seeping in and wetting the saw-dust.

The second method used is as follows: The pipe is covered with two layers of 1-in. hair felt, each layer firmly secured with copper wire. Between each layer of hair felt is placed roofing or asphalt paper. On the last job (on a bridge crossing a stream), the asphalt paper was sewed on to the hair felt and a great deal of time and labor was saved in placing this around the pipe. A metal casing of galvanized sheet iron is then placed around the pipe and this is held in place by ⅛-in. steel bands 1½ in. wide placed 2 ft. apart. The entire casing is then given two coats of paint to conform with the color of the bridge. Although the saw-dust method is much cheaper and answers the purpose, it is believed that the latter method is more satisfactory and the appearance of the complete job is much neater.

The cost of these jobs complete, exclusive of cost of pipe and fittings, is shown in Table I.

Marginal Lands for Impounding Reservoirs

In a discussion by W. H. Dittoe and G. Gale Dixon, respectively chief engineer and deputy chief engineer, the Mahoning Valley Sanitary District of Youngstown, O., in Vol. 93, 1929 Transactions of the American Society of Civil Engineers, the following table showing the acquisition of land about various important reservoirs is given:

Width of Marginal Lands Acquired About Various Impounding Reservoirs

Reservoir	Land acquired, in acres	Area flooded, in acres	Ratio of land acquired to area flooded	Length of shore line, in miles	Mean width of marginal strip, in feet
New York, N. Y:					
Ashokan	15,222	8,180	1.86	40	1,450
Kensico	4,500	2,218	2.03	30	630
Schoharie	2,372	1,145	2.07	16.5	610
Boston, Mass.:					
Wachusett	5,163	4,200	1.23	35	230
Providence, R. I.:					
Scituate	12,547	3,600	3.5	38	2,150
Akron, O.:					
Rockwell	2,065	769	2.7	20	535
Columbus, O.:					
Griggs	452	363	1.2	....	65
O'Shaughnessy	1,159	829	1.4	....	250
Youngstown, O.:					
Milton	3,344	1,657	2.0	23	606
The Mahoning Valley Sanitary District, Ohio:					
Meander Creek (proposed)	5,248	2,010	2.6	40	690

High Manganese Effluents from Idle Filters

Experimental Work at Columbia Heights Filtration Plant Shows That Manganese Is Redissolved from Sand of Idle Filters

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IN reviewing the analytical data for the past 2½ years at the Columbia Heights filtration plant, we found that the monthly composite samples of the plant effluent yielded manganese, in several instances, in amounts ranging from 25 to 200 per cent greater than that found in the raw water samples. Although these amounts were far below the tolerable limits, the increase was such as to suggest another source of manganese. Since the raw water contained relatively small quantities of the element, it seemed logical, to assume that some stage in the treatment of water was responsible for the increase of manganese. Because it is well known that manganese is deposited and retained on the sand grains of filters this seemed the most probable origin of its increase.

In this work we are dealing with a source of contamination in the plant effluent brought about by the resolution of the manganese oxides from the sand grains of idle filters. This condition is possible even though a raw water supply may contain only small quantities (less than the tolerable limits) of manganese compounds; because a gradual accumulation of the element takes place in filter beds where it may again be redissolved when conditions become favorable.

Experiments at Columbia Heights Filtration Plant.—The Columbia Heights filtration plant contains two sets of rapid sand filters. Filters numbered 1 to 16, inclusive, are as a general rule in service a greater part of the time, while the set of filters numbered 17 to 24, inclusive, have been used only in cases of emergency during the past two years. The latter filters had been out of service for many months and were utilized as a means of determining the extent to which manganese redissolves from the filter beds while in contact with wash water over a period of time. While remaining idle, the filters 17 to 24, inclusive, were washed periodically, about once every month during the winter months and more frequently during the warmer weather. During the month of May they remained idle, in contact with about 20 in. of wash water covering the surface of the sand. Twenty-five samples of water taken from the surface of these filters (about 1 in. above the sand) had a manganese content greatly exceeding the tolerable limits

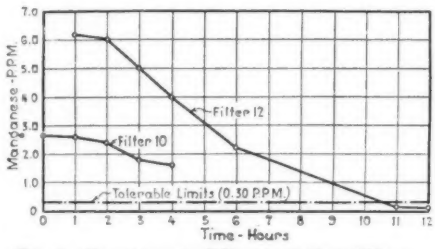


Fig. 1—Manganese in Effluents of Two Filters Idle 38 Days and Operated at a Rate of 1.5 M. G. D.

and in one sample 15.0 p.p.m. manganese were obtained. Four of the filters had been treated with 2 p.p.m. copper sulfate in order to check the growth of algae which had been troublesome in the past. In Table I the results obtained from these four treated filters, are compared with four untreated filters. It will be noted that the treated filters gave a lower manganese content than the untreated filters.

An additional matter of interest in this connection was to determine to what extent biological action influenced the dissolved oxygen content and the hydrogen-ion concentration. The water as applied on these filters had dissolved oxygen content of from 7.0 to 8.0 p.p.m. and an initial pH of 6.7. After the 31-day idle period samples of water from 3 filters gave zero dissolved oxygen, while the samples from the remaining five gave values below 1.5 p p.m. The hydrogen-ion concentration changed to a minimum of pH 7.1 and maximum of pH 7.6.

The next problem was to determine whether or not a filter which had remained idle and then washed would deliver manganese when placed into service. Hourly samples were taken from the effluents of two filters that had been out of service five weeks and were analyzed for their manganese content. The results obtained, as indicated in Fig. 1, show that these filters delivered water with a manganese content far in excess of the amounts delivered by filters in regular service. It will also be seen that the curves represent values greatly exceeding that of the tolerable limits.

In a series of samples an attempt was made to correlate the manganese content with the degree of reaction that takes place when the o-tolidine reagent is used. Table II shows the extent to which manganese in parts per million is related to the false chlorine readings in parts per million. Samples 2, 4, and 6 vary considerably in manganese concentration, but all apparently react to the same degree with o-tolidine reagent after a 10-minute contact period. Sample 2 exceeds that of sample 5 in manganese content by more than 1.5 times, yet the same amount of false chlorine is produced by both. In a comparison of sample 1 with sample 8, it is noted that a difference of 30 times in the manganese content exists while the difference in the false chlorine obtained amounts to only 7 times. The 50-minute contact period with the o-tolidine reagent gave values from 0.01 to 0.04 p.p.m. higher than that of the 10-minute period. However, one sample failed to show an increase of false chlorine.

Discussion.—Early investigators have attempted to explain the deposition and resolution of manganese in filter sands on the basis of physico-chemical action rather than by biolysis. Able support of the biological theory in bringing about a change of manganese from the soluble to the insoluble form and vice-versa, is presented by the independent work of Baylis and Kühr. Baylis demonstrated by laboratory experiments that the deposited manganese on sand will redissolve. He used samples of sand taken from filter beds which were covered with slime produced by bacterial growth, and submerged them in water for a week or more. The solution obtained from this sand contained about 10 p.p.m. soluble manganese. He repeated this experiment a number of times, and in every case when the sand was covered with slime there was an increase of soluble manganese. He considers that in covering the deposited manganese with slime, the bacteria

Table I—Manganese in Surface Samples from Treated and Untreated Idle Filters. Results in Parts per Million

Treated Filters*		Untreated Filters	
Number of bed	Manganese p.p.m.	Number of bed	Manganese p.p.m.
17	1.0	18	4.5
19	3.8	20	15.0
21	1.5	22	3 8
23	1.0	24	6.0

*Treated with 2 p.p.m. copper sulphate.

Table II—Comparison of Manganese in Parts per Million with False Chlorine Produced in the Sample*

Sample	Manganese p.p.m.	False Chlorine 10 minute contact p.p.m.	False Chlorine 50 minute contact p.p.m.
1	15.0	0.07	0.10
2	6.0	0.04	0.06
3	4.5	0.08	0.12
4	3.8	0.04	0.04
5	3.8	0.04	0.07
6	1.0	0.04	0.07
7	1.0	0.03	0.05
8	0.5	0.01	0.02

*Using o-tolidine made according to Standard Methods, 1925 edition.

produce conditions favorable for its being thrown into solution. Kühr, on the other hand, gives experimental evidence that manganous compounds can be oxidized to manganese dioxide by microbial growth. He demonstrated this fact by inoculating the surface of a solid, synthetic media containing manganous carbonate with bacteria and fungi as found in raw dune water and in the mud of rapid sand filters. The cultures developed a typical dark brown manganese dioxide. He concludes that "by the biological oxidation process the dissolved manganous compounds can be transformed into undissolved manganese dioxide, and that inversely by biological reduction, which always takes place in anaerobic processes, undissolved manganic compounds can pass into dissolved ones."

The fact that soluble manganese is retained as manganese oxides in active filters and redissolved in idle filters is supported by the results obtained in the experiments conducted. Soluble manganese found in 25 samples of water taken from the surface of idle filters seems to confirm the biological resolution theory, and the negligible amount or absence of manganese in the effluents of filters in regular service adds evidence to the known fact that retention of manganese compounds as insoluble manganese oxides occurs on sand grains. In Table I a comparison is made between the amount of soluble manganese obtained from a set of treated and a set of untreated filters. Probably this difference would have been even more marked had the treated filters received frequent and larger doses of copper sulfate. This phase of the subject will require further study in order that more definite and complete information can be secured. A suggestion is also seen for further investigation in the use of other sterilizing agents which would be applicable for filters out of service. If no subsequent operating difficulties arise another means of preventing the resolution of manganese may be found in permitting filters to remain dry during their idle period.

The conditions were favorable for anaerobic decomposition in the water on these idle filters is evident since the dissolved oxygen content in five filters was very low, and in three of the filters it was zero. Originally the applied water had a dissolved oxygen content of about 7.5 p.p.m. The logical deduction to be drawn here is that the depletion of oxygen was due to aerobic biological activity, since the loss of oxygen can be explained in no other way. Another condition probably brought about by biological action is the change found in the hydrogen-ion concentration from a pH of 6.7 in the applied water to an average of about 7.3 at the end of the 31-day period. It is possible that the end products of anaerobic decomposition increased the pH above the neutral point, as it is known that ammonia may be evolved from such decomposition. It would seem, therefore, that the resolution of manganese did not take place because of the presence of an acid condition.

The most interesting observations made are those represented by Fig. 1 showing the delivery of soluble manganese from two filters, 10 and 12, which had been idle for more than five weeks. The filters had been thoroughly washed and filtered to waste for 15 minutes before being put into service. At a filtration rate of 1.5 million gallons per day, the effluents of these filters continued high in manganese concentration over a period of several hours. Five samples were taken from filter 10 and analyzed. It delivered an initial manganese concentration of 2.7 p.p.m. Out of the seven samples analyzed from filter 12 the initial sample showed manganese to the extent of 6.2 p.p.m. In the latter filter, a sample analyzed after the second hour showed a slight decrease of manganese, and the subsequent hourly samples showed approximately 1 p.p.m. decline in the manganese content. A normal condition with respect to manganese was not obtained until a period of more than 11 hours had elapsed. The initial yield of manganese for filter 10 is only about one-half that of filter 12. Although more samples should have been taken from filter 10, it would seem from the curve plotted and also by interpolations that the period of time necessary for this filter to reach a normal output of manganese would be at least 8 hours.

In addition to the samples from the two filters shown in Fig. 1, one sample was obtained from filter 9, which had been out of service 42 days. This filter yielded a manganese content of 0.40 p.p.m. after it had been in operation nearly 7 hours. The delivery of a high manganese content does not result from filters which are in continuous operation, as evidenced by the analysis of the effluents of normal filters. The average manganese content of these filters, during the same period of time, rarely exceeded 0.05 p.p.m., and approximately 50 per cent of the time was zero.

Since the effluents of the idle filters delivered such an excessive amount of soluble manganese, it was desirable to know whether or not the plant effluent would show an increase over the monthly average. The monthly composite samples of the plant effluent for the years 1927 and 1928 gave a maximum manganese content of 0.06 p.p.m., a minimum of 0 and an average of 0.02 p.p.m. Two samples of the plant effluent were taken at about 22 and 29 hours after filters 10 and 12 were put into operation. It was calculated that sufficient time had elapsed to allow for a detention period in the clear water reservoir so that a representative sample could be secured. Analysis of the first sample, taken after 22 hours, showed a manganese content of 0.10 p.p.m. and of the second, at the end of 29 hours, of 0.08 p.p.m. While these figures do not represent amounts that are likely to cause trouble, it can be seen that a potential danger exists when a number of idle filters are put into service at the same time.

The reaction of manganese compounds with the o-tolidine reagent produces the characteristic color that free chlorine produces with the same reagent. In Table II no direct relationship was found to exist between the amount of manganese in parts per million and the amount obtained in false chlorine readings. Campion at the Barberton plant in Ohio gives experimental evidence to show that false chlorine readings reach their maximum value when the oxidation of manganese has gone to completion. Apparently the degree of oxidation in which the manganese exists is a factor in the production of the yellow color with o-tolidine. This may explain the lack of correlation in the values tabulated in Table II. Forsberg made the interesting observation that "Water containing manganese as manganic hydroxides gives the same reaction with o-tolidine as chlorinated water, irrespective of whether a water, alcohol, sulphuric acid or hydrochloric acid solution of the reagent is used." Aside from the deduction made from Table II, a point of interest is found in the fact that the majority of the false chlorine readings produced by manganese compounds increased with time, while on the other hand it is a well known fact that in the test for free chlorine the converse is true.

From a consideration of the bio-chemistry of the subject the deductions are that the deposition of manganese is caused by a biological oxidation process; and inversely, the biological reduction induced by anaerobic decomposition converts manganese oxides into the soluble manganous compounds. The layer of manganese oxide deposit formed on the sand grain serves to attract more soluble manganese, which in turn is oxidized by the aerobic microbes to the insoluble form. The process is continued until finally the sand grains are covered with many layers of manganese oxide.

In the case of incrustations formed on the sand grains of filters where alum is used as a coagulant, there exists, in addition to an insoluble manganese oxide and bacterial jell, aluminum hydroxide. The two latter substances may occur in intimate combination with the manganese oxide, or they may occur in alternate layers with the manganese oxide. The writers believe that when anaerobic decomposition has rendered the manganese oxide soluble, the coatings of bacterial jell and aluminum hydroxide probably act in a manner similar to that of a semi-permeable membrane, delaying the rate of diffusion of soluble manganese from the sand grain. It is therefore reasonable to suppose that, when a filter is washed, only the outer soluble manganese com-

pounds are rapidly removed. Following this reasoning it is probable that after the filter is placed in service, the hydrostatic pressure created is sufficient to draw the soluble manganese out from the inner layers of the same grain. This explanation may account for the delivery of a high manganese content over a period of 8 hours or more by idle filters which are again put into service.

Summary.—Analytical data are presented to show that manganese is redissolved from the sand of idle filters.

It is demonstrated that the effluents of idle filters will deliver a high concentration of soluble manganese, and that this condition may continue for a number of hours.

No direct relationship is found between the amount of manganese present in a water and the amount of false residual chlorine produced with o-tolidine reagent.

The information obtained thus far indicates that biological action is an important factor in the process of redissolving manganese.

A suggestion for the prevention of the resolution of manganese in idle filters may possibly be found in the proper use of copper sulfate.

Acknowledgment.—The foregoing is an abstract of a paper presented June 14 before the Minnesota Section of the American Water Works Association.

Contamination by Consumers' Water Uses

A New Viewpoint of Possible Water System Pollution

By S. B. MORRIS
Chief Engineer, Water Department, Pasadena, Calif.

IN reviewing water works literature, I do not recall any papers having been presented or restrictions drawn regarding piping and use of water by consumers after water has passed through the meter to their private premises. Before discussing the general possibility of such contamination, I should like to recite a few instances of contamination which have come under my observation.

How Lime Got Into the Mains.—One day we received a complaint of excessive lime in the water of a limited district. Upon investigation we found that a water softening plant where water was treated for boiler feed purposes, was the cause of trouble. An elevated tank with pressure normally much less than that on the public water system stored this heavy dose of lime. During an emergency shutdown of the water main, a check valve on this line failed to operate and the lime water contents of this tank was drawn into the distributing system.

Cafeteria Has Connection with Swimming Pool.—In another instance, we received complaint at a school cafeteria that excess chlorine in the water was discoloring the silverware. We knew this could not be the case, as the territory involved received no chlorinated water. Upon investigation, we found that an ortho-tolidin test showed about one-tenth part per million of free chlorine at the cafeteria, whereas a test from a fire plug directly connected to the city main adjoining the school showed no free chlorine. It was apparent that the cafeteria was receiving its water from a connection to the swimming pool circulation line, which proved to be the case. Apparently a serious epidemic was only averted by continuous excess chlorination of the swimming pool water.

Sewage Backs Up Into Domestic Water System.—At another time, we received complaint of sewage odor in the showers at a public swimming pool. We immediately investigated and found the complaint justified. There was a sewage pumping plant about 500 ft. from the swimming pool. At this plant horizontal, centrifugal pumps were used with automatic, intermittent operation raising the sewage from a concrete storage well up to the public sewers about 100 ft. higher in elevation. The sewage pumps were located above the elevation of the sewage in the storage well necessitating pumps operating under a suction. The pump operator had had so much difficulty in priming these pumps that he installed a hose connection between a garden valve connected to the public water supply and used this hose to prime the pumps. Finally he had wired one end of this hose to a stop cock on top of the pump. At the time of the complaint, he had forgotten to close the stop cock and the pressure on the sewage pump line being in excess of the domestic water supply, caused accrued sewage to back up into the domestic water system actually contaminating the shower baths at the swimming pool.

How Air Got Into the Water.—In another instance, we received complaints of air in the water over a considerable area. For several months we were unable to discover the source of air the consumers complained of. Finally, we found a water meter which registered backwards. Investigation showed that the machine shop and garage furnished by this service had an air compressor and air storage tank. In order to diminish the water bills, the proprietor of this shop had blown water back through the water piping, reversing the water meter while the air passed off into the water distributing system. Probably no contamination occurred through this operation, but it illustrates the possibilities of water being passed through the meter into the distribution system.

From time to time new industrial or sanitary equipment offering opportunities of contamination comes upon the market. The water utility has no direct knowledge of what equipment or piping is being installed by the individual consumer within his private premises. How is this matter to be controlled? I believe the only way is by co-operation with the building and plumbing inspection departments of the city.

Acknowledgment.—The above is an abstract of a paper presented Oct. 24 before the California Section of the American Water Works Association.

Sanitation Department Urged for New York City

The Merchants Association of New York City has opened a campaign in behalf of the proposed charter amendment which would create a new city department of sanitation for the city of New York.

Action by the association followed a study of the subject by the organization's committee on sanitation, public health and water supply, of which Prof. Olin H. Landreth is chairman. Letters were sent to thirty-eight other organizations. They declared in part:

"The measure relates to a vital matter—the health of the people of New York. The planning of adequate and comprehensive sewage and garbage disposal systems which will free our rivers and beaches from the filth with which they are now polluted, and thus rendered a menace to the thousands who use them, is one of the gravest problems faced by our city. Attempts to deal with this subject in the past have been fruitless of any large accomplishment.

"The state of New Jersey has recently instituted with the Federal government an action to prevent the dumping of garbage at sea—alleged to be a contributing cause of pollution affecting not only the New York but also the New Jersey beaches. The work of providing adequate disposal plants in order to anticipate the intolerable situation which might arise if the Federal government should put a stop to this practice should be speeded up.

"Sanitation experts and others who have studied this matter believe that existing agencies are inadequate to deal with the problem. Those who have given most thought to the subject are in agreement that the creation of a department of sanitation, such as is now proposed, offers the greatest expectation of obtaining expeditiously a sound and comprehensive plan of relief."

Sewage Sludge as Fertilizer

Sludge from Present Sewage Plants Could Furnish 8,000 to 10,000 Tons of Nitrogen Yearly

By WILLEM RUDOLFS

New Jersey Agricultural Experiment Station, New Brunswick, N. J.

HUMAN wastes and wastes from certain industrial processes contain considerable quantities of plant-food, which under ordinary conditions are lost. If the sewage is not treated, the material finds its way into streams and water courses; and in those cases where sewage disposal plants are present, the organic solids are retained as a sludge.

There are two general methods of sewage purification; namely, settling of solids followed by anaerobic digestion, and oxidation by forcing air into sewage. In both cases quantities of sludge are left behind. The sludges contain appreciable quantities of nitrogen and some phosphorus and potash, but the chief value of the sludge for fertilizer is in its nitrogen content.

There is at present only one sewage disposal plant in this country—the Milwaukee activated sludge sewage plant—where sludge is commercially marketed on a large scale. In a number of instances the sludge is locally sold for a nominal sum to farmers in the neighborhood. In the majority of cases the sludge is either buried or used for filling-in low places.

Composition of "Milorganite".—The product sold by Milwaukee under the trade-name of "Milorganite" has been dried, finely and evenly ground, and contains 5 per cent or less moisture.

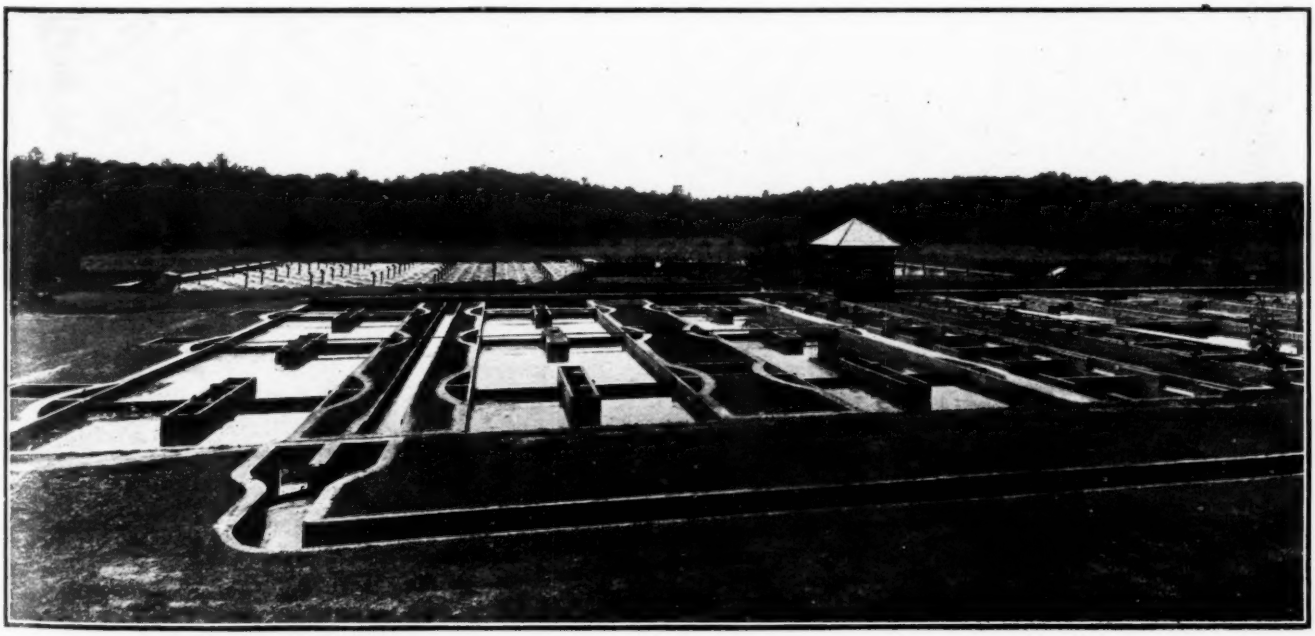
Kadish (2) has published an analyses of the material, representing 145 carloads shipped during a part of 1927. The composite sample contained:

	Per Cent
Total nitrogen	5.42
Water soluble organic nitrogen	0.30
Nitrogen insoluble in neutral permanganate	0.71
Active nitrogen by alkaline permanganate	3.22
Total phosphoric acid	3.08
Available phosphoric acid	2.43

Analysis of Plainfield Sewage Sludge.—There are only a few activated sludge sewage disposal plants in the country as compared with disposal plants where the sludge is digested anaerobically. Sewage sludge looses part of its nitrogen content in the course of digestion, probably most of it as nitrogen gas. As examples of the fertilizing value of fresh (undigested) solids, partially decomposed material, and fairly well-digested sludge, analyses of material obtained from the Joint Sewage Disposal Plant of Dunellen, Plainfield, and North Plainfield, N. J., are given in Table I. The analysis used for illustration show a rather high initial total nitrogen content. It usually runs at this particular plant between 3.50 to 4.00 per cent nitrogen. An average of a number of analyses shows 3.62 per cent total nitrogen in the fresh solids and 2.70 per cent in the ripe (digested) sludge.

Analysis of Sludge from 14 Cities.—At the request of the author, a number of superintendents and engineers in charge of sewage disposal plants in the country forwarded samples of sludge to our laboratory for analysis. The pertinent results together with some remarks are given in Table II. The percentage ash of the materials varies considerably. There are two reasons for this variation; namely, inorganic trade wastes present in the sewage, and the degree of decomposition accomplished. Domestic wastes settling out at a sewage treatment plant contain from 20 to 25 per cent ash. These solids, kept for a considerable number of days in digestion tanks, decompose under anaerobic conditions, producing large quantities of methane, some carbon dioxide, and some nitrogen gases (CH_4, 70 to 75 per cent; CO_2, 18 to 25 per cent; N, 3 to 10 per cent). The amount of ash remains constant but the percentage increases in relation to the percentage decrease of organic matter, with the result that well-digested sludge contains approximately 50 per cent ash. When inorganic trade wastes are received, the initial ash content varies from 25 to as high as 50 per cent, according to the amount of trade waste present.

The nitrogen content of the sludge depends upon the same two factors. The better digested, the lower the nitrogen content; and the more trade waste, the less the original percentage nitrogen, except when slaughterhouse, tannery, and such wastes are treated. If less than 2.5 per cent total nitrogen is present in the digested sludge there



Intrenchment Creek, Sewage Disposal Plant, Atlanta, Ga.

is ordinarily inorganic trade waste present.

The sewage plant operator wishes to produce a well-digested material, because this material will dry rapidly and has no odor, but in doing so, the fertilizing value of the sludge decreases.

The amount of dry domestic solids produced per capita per day is fairly constant. A large number of analyses shows it to be in the neighborhood of 70 gm. per capita daily. With trade wastes present the amounts rise sometimes as high as 200 gm. per person per day. The amount of solids retained at a domestic sewage treatment plant varies from 30 to 65 gm. per capita per day, depending upon the type of treatment plant. In the aggregate the quantities of sludge produced are enormous. Milwaukee alone, for instance, produces approximately 25,000 tons of dry sludge, which must be handled in a wet condition, requiring machinery to treat 500,000 tons sludge. Where the sludge is digested, the quantities are lower. Fuller and McClintock (1) state that at present about 15 per cent of the sewage is treated in this country. The total dry solids in the sewages received at these plants is between 400,000 and 500,000 tons per year. Assuming a minimum average of 2 per cent nitrogen in the sludge, the annual loss of nitrogen amounts to from 8,000 to 10,000 tons for the treated sewage only. Most of this could be used as fertilizer, since it is already collected at sewage disposal plants.

From 150,000 to 200,000 tons of nitrogen, as a conservative estimate, present in sewage is annually lost by discharging sewage into streams and water courses, and by wasting the sludges.

Summary.—The nitrogen content of sewage sludges varies with the type of treatment. Aerobically treated sewage sludge contains about 5 per cent, and anaerobically treated sludge averages about 2.25 per cent nitrogen. Only one sewage treatment plant markets its sludge on a large scale; at a number of places the sewage sludge is given away or sold for a small nominal sum. It is estimated that from 150,000 to 200,000 tons nitrogen is lost annually. Based upon analysis of a number of sludges from different sewage plants,

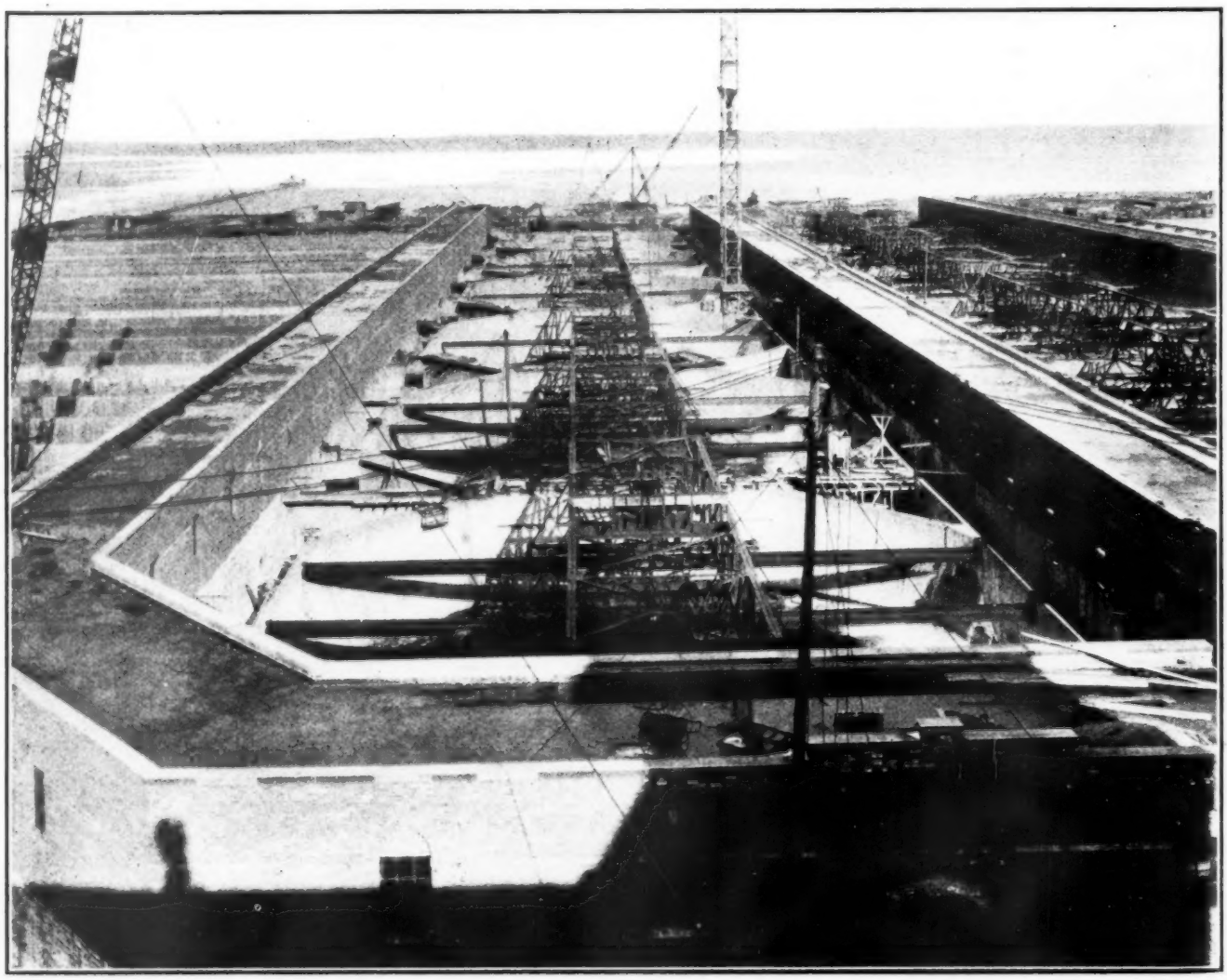
Table I—Analysis of Plainfield Sewage Sludge

	Fresh solids, per cent	Partly digested solids, per cent	Fair digested solids, per cent
Total N	4.81	3.98	3.55
Insoluble N	4.14	3.61	3.41
Soluble N	0.67	0.37	0.14
NH_3-N	.....	.....	0.09
Soluble organic N	.....	.....	0.05
Active insoluble N	0.98	1.89	1.57
Inactive insoluble N	1.16	1.71	1.84
Ether extract	15.11	7.79	6.83
Crude fiber	6.01	5.55	5.50
Ash	25.70	37.90	42.62

Table II—Analysis of Sludge from Different Disposal Plants

Place	Total N, per cent	Ash, per cent	Degree of digestion
Trenton, N. J.	1.18	51.7	Good
Rochester, N. Y.	1.96	59.1	Good
Schenectady, N. Y.	1.86	56.9	Fair
Schenectady, N. Y.	1.97	44.1	Poor
Fort Worth, Tex.	2.77	31.5	Poor
Fort Worth, Tex.	2.07	48.4	Fair
Marion, O.	2.70	48.1	Fair
Marion, O.	2.49	46.9	Good
Baltimore, Md.	2.48	32.1	Poor
Baltimore, Md.	2.18	50.5	Good
Portsmouth, O.	0.63*	81.9*	Much sand
Chambersburg, Pa.	2.53	33.2	Poor
Mount Alto, Pa.	1.70	41.2	Fair
Lebanon, Pa.	2.68	40.4	Fair
Worcester, Mass.	2.97	36.9	Fair
Chicago, Ill.	1.67	58.4	Good
Antigo, Wis.	2.09	55.2	Good
Plainfield, N. J.	3.16	37.2	Fair
Plainfield, N. J.	2.56	49.8	Good
Average	2.28	45.4	

*Left out of average.



Sewage Treatment Plant at Milwaukee, Wis., During Construction

it is estimated that from 8,000 to 10,000 tons nitrogen per year could be saved at present.

Acknowledgment.—The above is a paper of the Journal Series of the Department of Sewage Disposal of the New Jersey Experimental Station. It is reprinted here from Soil Science.

(1) Fuller, G. W., and McClintock, J. R., 1926. Solving Sewage Problems. New York. (2) Kadish, V. H., 1928, Milorganite—A new fertilizer material. Indus. and Engin. Chem. 20:9-10.

Sewage Plant Operations at Milwaukee

The following notes on the operation of the activated sludge plant of Milwaukee, Wis., are taken from the recently issued 1928 report of the common council on the activities of the city departments, boards and commissions:

Throughout the year 1928 the sewage disposal plant was operated at its full capacity. At the beginning of the year, the average amount of sewage treated was 85,000,000 gal. for 24 hours. Due to additions mainly in the suburban sewer system, the quantity treated rose during the year to an average of 95,000,000 gal. per 24 hours. During each day there is a peak load on the plant which amounts to from 130 to 140 million gallons in 24 hours.

During 1928 the total time of bypassing due to storms was 7.8 per cent of the total number of hours per year. Therefore, 92.2 per cent of all the sanitary sewage produced in the metropolitan district during the year was purified. the balance being taken care of by dilution in the large basin of Milwaukee bay.

Sales of Fertilizer.—The plant removes from the sewage all the suspended solids, which are converted into a commercial fertilizer and sold. The total output of the plant during the year amounted to 33,000 tons of fertilizer for which approximately $600,000 were received.

The demand for Milorganite, the fertilizer produced in the sewage disposal plant, has always been greater than the supply. About 70 per cent of the output is shipped to fertilizer manufacturers who blend it with other ingredients such as phosphates and potash and sell this product for use mostly in the states that grow cotton and tobacco. The remaining 30 per cent is sold for use directly without the admixture of any other material. This unmixed Milorganite is especially valuable for the growing of fine lawns and for general garden use. Most of this part of the output goes to golf courses, park boards, and other places where fine lawns are maintained. A considerable amount is sold in 100 pound bags to the gardeners of Milwaukee and the surrounding suburbs. These local sales in 1928 brought a return of approximately $10,000. The retail sale price is $1.50 per 100 lb. bag.

The research work was pursued vigorously during the year and has furnished valuable information which will be used in the plan of the necessary extension. The four small plants in the testing and control station were continually operated, and the results obtained were analyzed and tabulated.

During 1928 an elaborate cost keeping system was worked out which will be put into service at the beginning of 1929. This will make it possible to determine accurately the cost of every phase of plant operation.

The intercepting sewer system was somewhat extended during the year.

Elimination of Odor.—During 1928, the method used in treating odor, introducing chlorine gas into the dryer discharge, proved very successful. Very few odor complaints were received during the year, and these were carefully traced to their source. Some of the complaints were clearly caused by odors emanating from sources other than the sewage disposal plant, but several were found to be well founded and caused by faults in the operation of the dryer equipment. Continued experiments with this equipment has resulted in an almost entire absence of complaints.

When the chlorine treatment of dryer gases was introduced, it was considered only a temporary expedient, and other methods of deodorizing were investigated. It was found that chlorine was superior in its effect to other methods and lower in its operating cost. During the year the total cost of chlorine used for deodorizing was almost exactly $10,000.

Plans for Plant Extension.—The gradual increase in the amount of sewage coming to the plant points to the necessity of plant extension in the near future. Another factor contributing to this is the rapid development of the suburban territory lying within the metropolitan district. It is expected that these plans will mature during the year 1929 and that the necessary building operations will commence during the year 1930.

Uniforms for Meter Readers

A recent regulation of the Hackensack Water Co., Weehawken, N. J., provides that all employees whose duties require them to enter homes of consumers must wear uniforms, according to Nicholas S. Hill, Jr., president of the company. Inspectors, meter readers and collectors will be attired in dark gray whipcord with the initials "H. W. C.," in red, on the right breast, and will wear caps bearing the letters "H. W. C.," also in red. The purpose of the uniforms is to enable customers to identify water company employees, and to guard against unwarranted intrusion of homes by impostors who in the past have not infrequently posed as representatives of the company.

Athens Celebrates Opening of Marble-Faced Marathon Dam

The Marathon Dam, overlooking the immortal battlefield, where in 490 B. C., the Greeks overcame the conquering Persians, and built to provide the city of Athens with an adequate water supply, was officially opened Oct. 25, by the President of Greece, Paul Koundouriotis, according to dispatches received by Ulen & Co., New York City, the builders of the dam. Premier Venizelos and members of his cabinet, and other personages important in Greek governmental circles, as well as the foreign diplomatic body took part in the inaugural ceremony. A general holiday for Athens was declared.

Adequate water supply has always been a problem to the Greek capital, and as far back as 594 B. C. Solon enacted laws governing its consumption. Three years ago, Ulen & Co., signed a contract for the construction of the Marathon Dam and a new acqueduct tunnel 8½ miles through the base of Mt. Pentelikon, and together with the Bank of Athens formed the Societe Anonyme Hellenique des Eaux des Villes d'Athens, Piree et Environs to manage the water system of the city of Athens and Piraeus. The Marathon Dam is the first important step in this project. Through repairs and improvements of the old Hadrian Aqueduct, built 2,000 years ago, the city is now furnished with 14 gal. per capita daily instead of 3½ gal. per capita. During the year 1930 according to the engineers, the new aqueduct will raise the supply to 135 gal. per capita per day.

The dam itself is the gravity section arch type, faced up and down stream with the Pentelikon marble of which the classic structures on the Acropolis are built. It is filled with solid concrete. Measured along the crest, the dam is 935 ft. long, with a thickness at the base of 154 ft. and at the crest 15 ft. Its height above the stream bed is 177 ft. More than 40,000 tons of Portland cement and 10,000 tons of pozzulan cement were used. The reservoir formed by the dam will impound 10,825,000,000 gal. of water, and even if this supply was not added to, it would be sufficient for the needs of Athens for two years.

The water for the reservoir is drawn from the hills above the plains of Marathon and will be conveyed through the tunnel 8½ miles long and then through pressure pipes to the city proper. In order to relieve the meager supply the engineers have also constructed a salt water system which pumps water from the Aegean Sea for the sprinkling of the city streets.

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Is the Boulder Dam Project on Colorado River Premature?

While there is little doubt that a large part of the Colorado River should be conveyed to Los Angeles County and neighboring territory eventually, we are by no means satisfied that work on this diversion should begin at once.

In 1924 the consulting engineers estimated that in 1950 the city of Los Angeles would have a population of 2,180,000, of whom 80 per cent would reside in 60,000 acres having at least 14 people per acre. They estimated that these 1,740,000 people (who will be designated hereinafter as urbanites) would use 130 gal. per capita per day for domestic purposes, plus about 32 gal. additional for industrial purposes. We deduce the 32 gal. from their estimate of 62,500 acre-feet of water annually used industrially in an area of 25,000 acres outside the 60,000 acres of urbanite territory, their estimate being 2.5 ft. of water per acre of industrial land. The combined domestic and industrial consumption of the 1,740,000 people in 1950 was estimated at 316,500 acre-feet, to which was added 248,000 acre-feet for the remaining 20 per cent of the population who will be designated here as "farmers." If the "farmers" used 130 gal. per capita per day for domestic purposes, that would take about 63,000 acre-feet, leaving 185,000 acre-feet for irrigation, or about one-third of the total 564,500 acre-feet used within the corporate limits of Los Angeles in 1950. For the farm districts the engineers estimated 2 ft. of water per acre per year.

Continuing the estimates of the consulting engineers, we have the following "Water requirements of the metropolitan area of Los Angeles County including city of Los Angeles in 1950."

	Acre-feet
Domestic use by "urbanites"	386,000
Industrial use	150,000
Domestic irrigation and other uses by others	823,000
	1,359,000

This last item is rather misleading, for by placing the word "domestic" first it produces the impression that "irrigation" is subordinate in importance, whereas as above shown, about 75 per cent of this item in Los Angeles city is for irrigation. If we take 75 per cent of 823,000 acre-feet as being for irrigation we have 627,000 acre-feet, which is nearly half the total estimated water consumption in Los Angeles county in 1950.

This immediately discloses that the proposed increase in water supply for the Los Angeles district is quite as much a farming project as a municipal project.

We believe that a similar analysis of the proposed uses of the Colorado River water elsewhere in Southern California will disclose that much more than half the water is intended to irrigation. While there is nothing wrong in such a plan, we doubt whether many of the city residents realize that they are being asked to vote bonds that are to be mainly used for developing farms.

Los Angeles county and nearby areas are already great agriculturally, and it may pay to develop them even more intensively, as producers of citrus fruits, grapes, etc. But judging by the wails that have repeatedly come from the growers of those products, overproduction has already made it difficult for them to reap a satisfactory profit. If with water lifts of only 100 ft., a farmer finds his profits small, what will they be when he has to pay for a 1,500-ft. lift over the divide between his farm and the Colorado River?

This is an economic question that we have never heard asked before, but it is certainly one to which both the rural and the urban residents of southern California are entitled to a definite answer.

In an article elsewhere in this issue data are given showing that at least 180 second-feet of additional water can be brought from the Mono Lake basin to Los Angeles. This would supply about 720,000 people with 160 gal. each per day. The cost of securing this additional supply is roughly estimated at $30,000,000.

We shall not be surprised if a study of the water resources of the Mono basin and the Walker River basin to the north of it, will show that 2,000,000 more people can be adequately and economically served from these regions of the eastern slope of the Sierra Nevadas, provided that the water is not used for irrigation but only for domestic and industrial purposes, and provided that none of it is wasted in developing water power at times when it should be stored for future domestic use.

Some years ago the editor appraised the hydroelectric power plants and water rights of the Southern Sierras Power Co. and subsidiaries in the Mono and Owens River regions, and thus gained considerable knowledge of the water resources of that region. He has visited that section many times in recent years. His belief is that the water resources of the Mono basin and the Walker River region to the north, may prove to be great enough to provide for the growth of metropolitan Los Angeles for another decade and still leave time in which to build the Colorado River aqueduct. All this provided that a great irrigational development of the Los Angeles district is not undertaken in the meantime.

As the article on the Mono basin project states, about $30,000,000 will be needed to acquire lands and water rights in that region, and to build reservoirs and aqueducts that will feed the Mono waters into the present Los Angeles aqueduct. While this is a large sum, it is dwarfed by the proposed Colorado River project. A vastly greater sum than $30,000,000 would be justified if the Colorado River project could be postponed 10 years.

The cost of the dam and power plant for that project is estimated at more than $150,000,000 and the aqueduct at about a like sum. Since great projects of this sort usually exceed the estimated cost, it would not be surprising were the entire project to cost $400,000,000 or more. It would seem needless to say that such an undertaking should not be lightly entered upon. Yet thus far it seems not to have had half the engineering study that it calls for, before even deciding to spend $300,000,000, to say nothing of deciding as to how long that construction can be wisely deferred.

We know that a great deal of important surveying and other engineering work has already been done, that many preliminary estimates have been made, and that the project is feasible. But what engineer is willing as yet to stake his reputation on what the project will cost?

In a recently published paper by Mr. R. H. Moulton, of Los Angeles, on "Financing Control of Water," we read: "Los Angeles and Southern California is again faced with a water crisis looking ten years ahead. The only conceivable plan existing at this time to adequately care for the future, will result in the building of the Boulder Canyon Dam by several states and subsidized by the Government, which will cost between $150,000,000 and $200,000,000. In addition to this, the aqueduct must be built several hundred miles, which will cost between $150,000,000 and $200,000,000."

What we wish here to emphasize is the indefiniteness of such cost figures. Between the $300,000,000 and the $400,000,000 there is a spread of $100,000,000, a difference sufficient to build a waterworks system to supply 1,250,000 people in the typical American city. We insist that where such vagueness exists as to the probable cost of a project, a great deal of engineering work remains to be done on it. But almost as important is the necessity of a most thorough study of other projects that may serve to postpone the enormous investment that the Colorado River project will involve.

If it be argued that at least half the investment will be self-supporting through the sale of industrial power, this too calls for careful study. To begin with there is a fair prospect that before the expiration of the bond amortization period of 50 years, there will be no power available except for pumping the water over the "divide." Little enough is known as to the probable rate of silting up of the proposed dam, but that little is ominous.

The Colorado River project is amazingly bold; unquestionably feasible and certain to be carried out eventually. But it is not like the purchasing of certain advertised goods whose makers' cry is, "Eventually—why not now?" To build this project now may be a great mistake, and we contend that until other projects are thoroughly examined it is likely to be a mistake.

We repeat a suggestion that we made several months ago: Many entirely separate groups of consulting engineers should be employed to study all the possible alternatives involved in this very complex problem. The fees of all of them combined will weigh as nothing when contrasted with the saving that some one of them is likely to effect. Parsimony in the hiring of consulting engineers may not account for the prospective abandonment of the San Gabriel dam site after spending $3,000,000 there; but it is fairly certain that had more consulting engineers been employed, some one of them would have pointed out the grave risk of running into unsuitable rock beneath the dam. Months before the rock slide at the dam occurred, M. M. O'Shaughnessy, City Engineer of San Francisco, said: "I understand lately the County Board of Supervisors of Los Angeles, in the plentitude of their political wisdom —after awarding a dam contract—dispensed with the Board of Consulting Engineers who had been employed on the San Gabriel Dam, a work involving an expenditure of $25,000,000. I believe this is a daring act of ignorance on their part, and a consulting board of wisdom and experience should be retained to supervise all features of the construction of this extraordinarily large project. In San Francisco we spent over a quarter of a million dollars hiring the best consultants in the United States to pass on the merits and features of our water project, and I believe it was the wisest money ever spent by the city."

If one board of three consultants was none too many on the San Gabriel Dam project, a dozen such boards would be no more than adequate for the Colorado River project and the other projects that may serve to postpone going to Colorado River. When we suggest several independent boards we have in mind the desirability of avoiding falling into ruts. Where a large number of men work on a board, usually a "we too" tendency develops, whereas what is really needed is more individualism. Moreover, competition among engineers, as among any other class of men, is the best of spurs.

The Rock-Fill Dam as a Safe Structure Where Bed Rock Is Faulted

After spending $3,000,000 in preliminary work and excavation, the proposed $25,000,000 dam in San Gabriel Canyon, near Los Angeles, will probably be abandoned. The faulted condition of the bed rock has become more evidently serious at greater depths.

The failure of the St. Francis dam, probably as a result of movement of the earth's crust along a fault beneath the dam, is still vivid in the memory of Californians. Hence the recent large slide of rock during the excavation of the San Gabriel dam foundation may spell abandonment of that dam site. Yet we raise the question whether this very favorable site should be abandoned. While we doubt the wisdom of building a concrete dam at that site, we believe that a great rock-fill dam could be safely built there.

As many of our readers know, the rock-fill type of dam consists of uncemented rocks sheeted on the upstream face with an impervious sheet of wood or metal. It resembles an earth dam in its massiveness, but differs in being permeable to any water that leaks through the sheeting. Consisting mainly of large stones, leakage streams do not cut the dam away as would occur if it were made of earth. Indeed, its very porosity is a safeguard, for no water can accumulate in its interior.

In a site where faulted rock exists, the rock-fill dam is unquestionably the safest type, if there is any likelihood that a slip may occur along the fault line. In a district subject to slight crustal movements, no fault should be regarded as a "dead fault." Geology is not yet such an exact science that a distinction can be drawn between "live faults" and "dead faults," other than one as to frequency of movement. Given a sufficient period of time, and it is probable that every "dead fault" will come to "life" again, just as many an "extinct" volcano has suddenly ceased being extinct.

The editor has found that there are 12 molton whirl centers in the earth, and that they are still active. Since these molten whirls are caused by the same electromagnetic forces that cause cyclonic storms, earthquakes are most frequent when great storms are most frequent. There is one great sunspot, storm and earthquake cycle about 81 years between "peaks," the last peak being about 1870. There is another similar cycle of about 161 years, its last peak being in 1778. There is another cycle of about 322 years, its last peak being in 1675. Halfway between these peaks, and often at intervals of one-sixth the length of these cycles, great weather and crustal disturbances occur. And the same holds true of the 11.2-year cycle, whose last peak was about a year ago, and whose weather and earthquake effects have been so pronounced at intervals during the past 18 months.

About 1939 and 1951 great crustal disturbances will occur, for those are the peak years of the 81 and the 161-year cycles. A fault that has been "dead" since 1871 may suddenly become alive in 1939 or sooner. So the geologist who bases his decision as to "deadness" of a fault upon evidence less than half a century old risks not only his reputation but the lives of all who live within the flood flow from a bursted masonry dam built over a "dead fault."

The only types of dam that have yet been built that invariably have resisted such crustal movements are those of earth or of loose rock. We have yet to hear of a dam of those types that has been destroyed by a crustal slip. Whereas, for reasons given at length in our issue of May, 1928, the St. Francis dam undoubtedly was destroyed by a slight movement of the rock beneath it along the line of an old fault. Other mysterious failures of masonry dams may have had similar causes.

California is the original home of the rock-fill dam, we believe. At any rate, there are many such dams in that state, and few in other states. Originally designed as an economic way of impounding water for placer mining use, it soon came into use for irrigation reservoirs. Later it was used by hydroelectric power companies, notably the Southern Sierras Power Co. and the Nevada-California Power Co., whose dams were studied by the editor a few years ago during an appraisal that he was making of the properties.

To the engineers of California we recommend a careful study of all the rock-fill dams in that state, with a view to using that type of dam wherever an otherwise good dam site is marred by faults in the bed rock.

Announcement of Articles on the Electron Theory of Weather and Allied Phenomena

In October a series of articles on the electron-whirl theory started with one on "Five Important Weather and Earthquake Cycles" in Water Works and Sewerage, and one on "The Periodicity of Geological Epochs" in Engineering and Contracting. This series will contain 16 or more articles based on the same theory, the articles relating to rainfall and air currents appearing in Water Works and Sewerage, while those relating to the geological and astronomical phases of the electron-whirl theory will appear in Engineering and Contracting.

The researches on which these articles are based have taken nearly all of the writer's time during the past three years. Starting as an inquiry into the relationship between sunspots and weather, the study broadened when it was found that in the neighborhood of certain centers of permanent cyclonic air whirls there is clear evidence of permanent whirls of the molten matter beneath the earth's crust. Later it became evident that the vagaries of the magnetic needle are largely accounted for by these same molten whirls, even to the slow oscillation of the magnetic poles. Since the molten whirls obviously affected topography, it became necessary to study the location of mountain chains and islands and the shapes of continents, in relation to these whirls; and from this gradually emerged the conception that 12 great molten whirls have been the main architects of the earth's crust. Never was the unity of nature better illustrated than in the correlation of phenomena seemingly so different as are sunspots, earthquakes, storms, and terrestrial lines of magnetic force. Aurora and magnetic storms had been previously correlated with sunspots, and one astronomer, Jerome Ricard, had been using sunspots as a means of predicting rainfall. Huntington had found a relation between barometric pressure and sunspots. Douglass had shown that radial tree growth often discloses an eleven year cycle corresponding to a well known sunspot cycle. De Geer had found the same cycle in the varying thickness of the annual sedimentary laminae ("varves") in ancient glacial lakes. Indeed, no small array of facts relating to the weather periodicity were available; but they were bound together by no definite theory susceptible of proof by an appeal to the laws of probabilities. Astronomers were in doubt as to the cause of sunspots. Meteorologists were in no doubt as to the cause of cyclonic storms, but unfortunately they were wrong, for their theory was in conflict with many facts. Geologists were in disagreement as to the cause of crustal movements and were not disposed to see much significance in facts that pointed toward their periodicity. Physicists had given but little thought to the electron as a cosmic force, and almost no thought to it as a force capable of causing weather changes. In general, astronomers ignored weather, weather men ignored astronomy, and physicists almost ignored both of those fields as realms of electron activity.

Under these conditions, it seemed to the writer that an engineer might be capable of coordinating facts and theories in these three realms of science; for since engineering consists in applying science, an engineer is not only capable of understanding any science, but of drawing sound inferences from scientific data.

Engineering is such a cosmopolitan profession that its followers are engaged in every sort of scientific work, from astronomy to farming, from the most purely "pure" to the most purely practical use of organized knowledge.

We take this occasion again to urge upon engineers the cultivation of a greater confidence in their ability to fathom nature's mysteries, regardless of the artificial realms in which scientists locate them. Nature is not a congeries of entirely independent classes of energy, but is a vast array of phenomena that are closely kin and probably identical as to the ultimate units whose varied motions and groupings produce this infinite variety that we call the universe.

H. P. Gillette

DISTRIBUTOR NEWS

The Distributor's Department in the Gillette Construction Group

South American Distributors Visited by American Bosch Executive

Mr. Herbert E. Fenner, General Service Manager of the American Bosch Magneto Corporation at Springfield, Mass., sailed from New York recently to make an extended tour of inspection of all the principal cities, and many smaller centers, in the countries of South America.

One of Mr. Fenner's principal duties will be to call upon the branch houses and distributors of those American manufacturers who use American Bosch precision electrical equipment as standard equipment on automotive products of their manufacturer. He will also visit official American Bosch service stations situated throughout the South American continent to make a first hand study of methods and offer advice and suggestions.

Gardner Denver Takes Larger Quarters in New York

Due to recent increases of business in the New England states, the Gardner-Denver Co. last week moved its New York branch office from 30 Church St. to larger quarters at 147 Varick St. The new quarters afford space for both office and warehousing purposes.

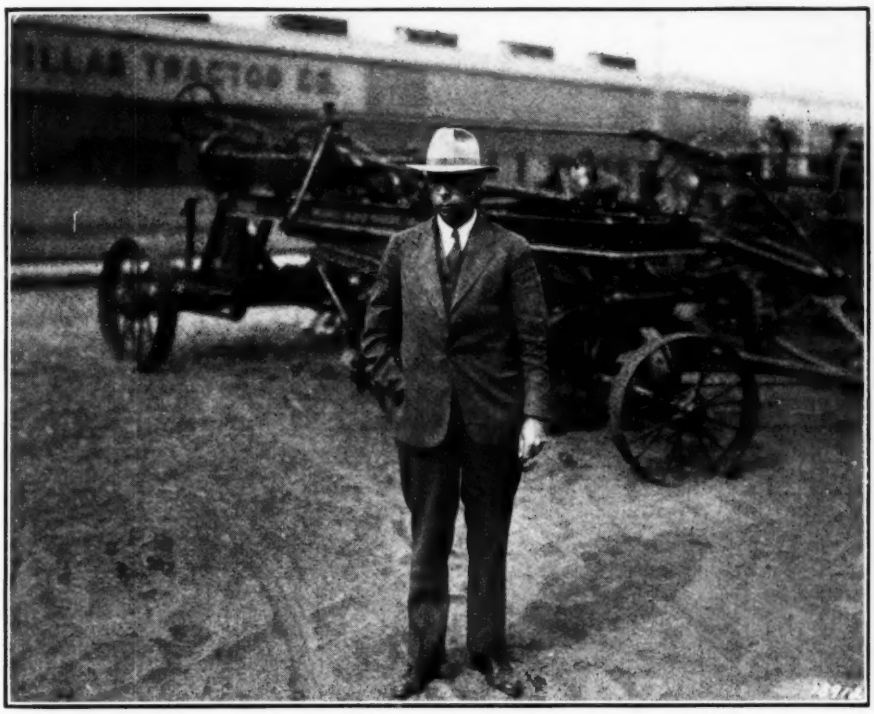
The Gardner-Denver Co., manufactures slush pumps, air compressors, high pressure drill steel forges, rock drills and drill sharpeners. Its plants are located in Denver, Colo.; Quincy, Ill., and LaGrange, Mo.

Chain Belt Opens Boston Office

The Chain Belt Company, Milwaukee, has opened a New England district office in Boston at 950 Park Square Building. This makes the eighteenth district office the company has opened in the United States within a comparatively few years. J. K. Merwin is district manager.

Mr. Merwin is a Yale University graduate of 1924. He worked at the Chain Belt Company's Milwaukee and West Milwaukee plants. Later he went to Cleveland with the company's subsidiary, the Stearns Conveyor Company.

The Boston office will manage the distribution of the company's entire line of Conveying Systems, Power Plant Equipment, Chains and Transmission Machinery.



Mr. Charles A. Spears, Now Representing "Caterpillar"

C. A. Spears Joins "Caterpillar"

C. A. Spears, widely known on the Pacific Coast as an authority on roads and streets, and who has a large acquaintance among contractors and governmental men, has joined the sales engineering staff of the Caterpillar Tractor Company, according to recent announcement. For the past 17 years Mr. Spears has been connected with Spears, Wells Company.

Announcement states that Mr. Spears' time will be devoted to helping solve earth moving problems confronting engineers, contractors and government officials.

Cutler-Hammer Establish St. Louis Warehouse

A warehouse, where a stock of standard C-H motor control and wiring devices will be carried for immediate delivery, has been established by Cutler-Hammer, Inc., at 1914 Washington Ave., St. Louis, Mo. The St. Louis district sales office has been moved from 611 Olive St. to the same address as the warehouse. The new location will give about 4,000 sq. ft. of space for warehouse purposes, and the increased facilities, it is announced by the company, will enable them to better serve their trade in the south and mid-west.

The territory served by the St. Louis office includes the states of Kansas, Missouri, Oklahoma, Arkansas, Louisiana, Texas, southern Illinois, southern New Mexico and parts of Tennessee, Kentucky and Indiana. Headquarters of Cutler-Hammer, Inc., are located in Milwaukee, Wis.

Wagner Electric Announces Branch Office Removals

Wagner Electric Corporation, whose headquarters are located at 6400 Plymouth Ave., St. Louis, announce that the Milwaukee sales office and service station, formerly at 501 Broadway, has been moved to 525-27 Broadway. The St. Louis sales office has been removed from 505 Sheel Building to 909 Plaza Olive Building.

According to announcement, the changes were necessitated by increased business requiring more space.

Change of Address

The Wallace Equipment Company of Detroit, Mich., announces that they are now occupying their new office, shop and warehouse located at 8511 Livernois Ave. The officials of this company are Mr. Geo. T. Wallace and Mr. John Cavan.

Bauer-Wach System Under License to DeLaval

The De Laval Steam Turbine Company of Trenton, N. J., builders of geared steam turbines for numerous weil-known vessels of the American Merchant and Naval Marine, has taken a license to manufacture under the American Bauer-Wach patents.

These patents cover the use of exhaust turbines and gears to utilize the exhaust from reciprocating engines, thereby developing and transferring to the same shaft 20 per cent to 25 per cent more power from heat otherwise

Dr. Gustave Bauer, Noted Engineer

wasted, as the turbine is more efficient at low pressures, and can use the greater volumes to which the steam expands under high vacuum. The fuel consumption of present existing plants can be reduced from 20 per cent to 25 per cent or an increase of power obtained as high as 25 per cent. This system also has the advantage that the turbine alone can propel the vessel at 60 per cent of full speed if the engine becomes disabled.

While the first installation of the Bauer-Wach System was made less than three years ago, it is said to have been successfully applied up to the present time in over 135 vessels with a total horsepower of over 500,000. Dr. Gustav Bauer, whose name it bears, is the engineer responsible for the machinery of the North German Lloyd steamer "Bremen," holder of Atlantic records for speed.

Eastern Manufacturer Returns from Southwestern Trip

Mr. Earle S. Philips, vice-president of the Good Roads Machinery Company of Kennett Square, Pa., has recently returned from an extended business trip into the southwest section of the country, his objctive being San Antonio, Texas. Mr. Philips reports conditions very good indeed in the southwest, and he is anticipating a good fall and winter business from that section.

During the trip Mr. Philips visited the Muskogee Iron Works Company of Muskogee, Okla.; J. B. Harbison Equipment Company of Little Rock, Ark., and C. P. Hollingsworth, San Antonio, Texas.

H. H. Curtice Heads AC Spark Plug Company

At a recent meeting of the Board of Directors of the AC Spark Plug Company of Flint, Mich., Harlow H. Curtice was elected president and general manager to succeed Basil W. deGuichard, whose resignation was due to ill health.

Just past his twentieth birthday and fresh from Ferris Institute, where he completed a commercial course, back in 1914, Mr. Curtice obtained his first position in the accounting department of the company to which he has now been elected president. His initiative in instituting improvements resulted in his becoming comptroller when he was twenty-one years old, making him, it is said, the youngest executive in the automotive industry.

In 1923 he was made assistant general manager, in which position he gained prominence in factory operations and management work. In 1927 he was given the title of vice-president. Through his management of the affairs of the AC plant, he not only came in contact with every department of his own company, but came in touch with practically all manufacturers in the automotive, aviation and marine industries.

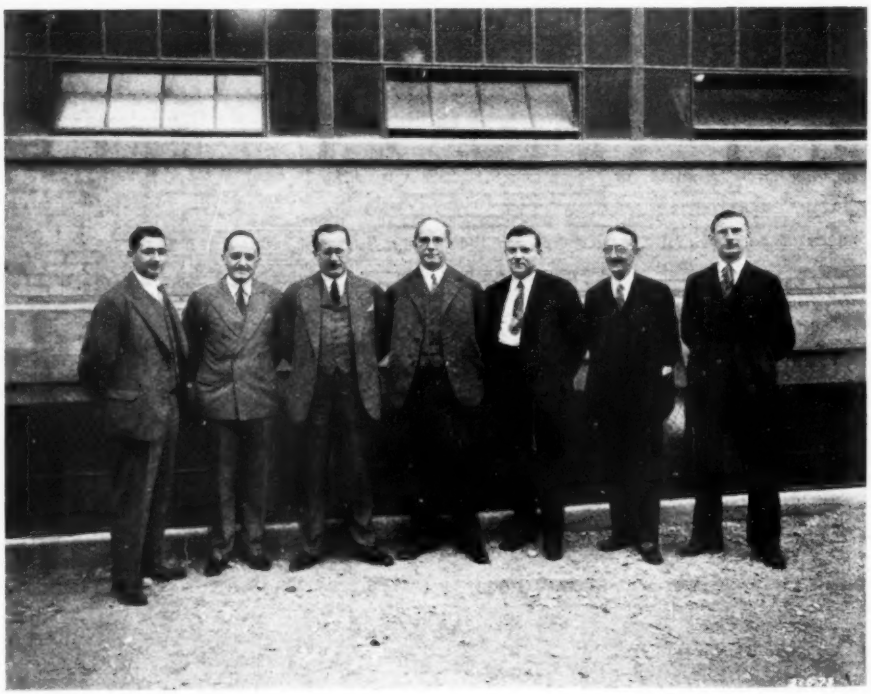
Mr. Curtice's election to the presidency of his company at the age of 36 years places him in the position of one of the youngest major chief executives in the automotive industry. He was born at Eaton Rapids, August 15, 1893, and graduated from the high school before entering Ferris Institute.

Gurley Sales Office Announces Removal

W. & L. E. Gurley, manufacturers of surveying instruments and hydraulic measuring equipment, announces the removal of their New York sales office from 25 Warren St. to 1202 Emigrant Industrial Savings Bank Building, 49 Chambers St.

The company believes they have been very fortunate in their new location, as the office overlooks City Hall Park, commanding a splendid view, and offers an unusual opportunity for the comparison of telescopes.

Mr. C. C. Covert, hydraulic engineer, is in charge of the display of instruments.



Foreign Industrialists Visit American Plants

A group of representatives of prominent German industrial concerns while on a tour of observation, visiting automotive plants, studying American manufacturing methods, and devoting much attention to highway construction and fuel problems. Picture taken at Flint, Mich., during visit at AC Spark Plug plant, a division of the General Motors Corporation.

Left to right: E. F. Kress, engineer with American branch of I. G. Dye Trust; T. G. McDougal, AC vice-president in charge of foreign plants; Richard Baacke, director of I. G. Dye Trust; W. Ranft, German Gasoline Company; Dr. Martin Muller-Cunradi, I. G. Dye Trust; Wa. Ostwald, editor of the Automobil-Technische Zeitschrift; Dr. Hans Navratiel, petrologist, AC Spark Plug Company.